ENGINEERING NEWS
AND
AMERICAN RAILWAY JOURNAL.
VOL. XLV. No. 1.

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THE INTERSTATE PALISADE PARK COMMISSION, appointed by the Governors of New York and New Jersey in June, 1900, for the purpose of selecting, locating and managing certain lands along the Palisades of the Hudson River for an interstate park, has made a preliminary report to the following effect: New York appropriated $10,000 and New Jersey $5,000 for the expenses of the several original commissions now acting as an interstate commission. It was resolved that the New Jersey appropriation should be devoted to paying for necessary surveys and maps of the Palisades, title, searches, etc., and the New York appropriation should be reserved intact for use as the nucleus of a fund for stopping blasting on the Palisades, which the commission wished to accomplish at the earliest moment. As the result of long negotiation, the commission now reports that it has utilized these appropriations as above set forth. Blasting ceased on Christmas at the only quarry now in operation and the largest one. This property has been purchased by the commission for $132,500, of which sum certain gentlemen of New York city have agreed with the commission to privately contribute $122,500 ($10,000 having been paid), provided that the legislatures of New York and New Jersey would this winter appropriate enough money to enable the commission to acquire all the property between Fort Lee and Huylers Landing, opposite Yonkers, and to construct a driveway at the foot of the Palisades. Options have been secured by the commission upon this remaining property on exceedingly reasonable terms, the vertical face of the cliffs to be donated to the commission outright. As will be seen from the above report, the commission has expended no money whatever for office rent, clerk hire or incidental or personal expenses.

THE LICENSING OF ARCHITECTS is under consideration in New York and a convention of the architects of the state has been called, to be held at the Architectural League Rooms in New York city, Jan. 19. A special committee of the League will report upon a bill which it has drafted for presentation to the legislature. The bill is framed on similar lines to the Illinois law (Eng. News, 2d Vol., 1897, p. 268). A similar bill passed both branches of the New York legislature in 1894, but failed to secure the Governor's signature.

SLOW SAND AND MECHANICAL FILTRATION for Washington, D. C., will be discussed at the Waldorf-Astoria Hotel, in New York city, on Jan. 4, 1901, before members of the Senate Committee on the District of Columbia. It is expected that several district officials and a number of specialists in water purification will be present.

PUBLIC CONTROL OR MUNICIPAL OWNERSHIP of street railways in Illinois are alternatives authorized by a carefully drawn and comprehensive bill submitted to the city council of Chicago on Dec. 17, 1900, by the street railway commission of the council. The plan for municipal ownership is novel in that it limits bonds for street railway construction to a lien on the street railway property, instead of having the lien embrace the whole credit of the city. Mr. Milton J. Foreman is chairman of the aldermanic commission, and Mr. Geo. C. Sikes (who is not a city official) is chairman.

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FOR RAISING THE "MAINE" in Havana Harbor, five bids were opened in Havana, on Dec. 19. Mr. John T. Cavanagh, of Boston, Mass., bid $735,000; Mr. Frank Pidgeon, of New York, bid $639,000, and the lowest bid was from the Schwartz Foundry Co., which offered to do the work for $80,000. Some of the bids were irregular in form, and the lowest one especially did not seem to note the government prohibition of the use of dynamite in removing the wreck. Mr. Cavanagh proposed to build pile piers and lift the wreck bodily by jack screws, and to transfer it to the Havana floating dock for such destination as the government might see fit. All bids have been rejected, and new bids are called for, to be opened on Feb. 1, 1901. The successful bidder may retain the ship, must finish his work by July 1, 1901, and must furnish a $25,000 guarantee.

A NATIONAL NAVAL RESERVE has been planned by the board of naval officers of which Admiral Dewey is the head, the other members were Rear-Admiral Crowninshield and Capt. R. D. Evans, U. S. N. This board proposes a minimum force of about 6,000 trained men and a maximum of 20,000 men from which the regular navy may select in case of emergency. The annual appropriation would be $100,000, which is to secure the enlistment of men for five years. The 600 officers of the reserve shall rank with but after regular officers of the same grade. Rank would be determined by examination before a board of regular line officers; and the list wold be open to naval officers who have resigned, to navigating owners of yachts, and masters of ocean-going steamships of over 3,000 tons displacement. The "retaining fees" paid to officers and men of this reserve would be as follows: Lieutenants, $200 a year; lieutenants, junior grade, $150; ensigns, $100; enlisted men, not to exceed an average of $50 per year. These officers and men must engage in annual drills on naval vessels, for not less than six continuous days and for an aggregate of not less than 30 days in a year. On the certificate of the commanding officer of the vessel, that the term of service is completed, the retaining fee and the proper pay for the time served will be paid over. Men so enrolled shall be regarded as in the service of the United States, to be called by the President into active service in case of war. Failure to serve when called out shall subject the person or persons to the penalties of desertion. They would be exempt from jury and militia duties.

THE INTERSTATE PALISADE PARK COMMISSION, appointed by the Governors of New York and New Jersey in June, 1900, for the purpose of selecting, locating and managing certain lands along the Palisades of the Hudson River for an interstate park, has made a preliminary report to the following effect: New York appropriated $10,000 and New Jersey $5,000 for the expenses of the several original commissions now acting as an interstate commission. It was resolved that the New Jersey appropriation should be devoted to paying for necessary surveys and maps of the Palisades, title, searches, etc., and the New York appropriation should be reserved intact for use as the nucleus of a fund for stopping blasting on the Palisades, which the commission wished to accomplish at the earliest moment. As the result of long negotiation, the commission now reports that it has utilized these appropriations as above set forth. Blasting ceased on Christmas at the only quarry now in operation and the largest one. This property has been purchased by the commission for $132,500, of which sum certain gentlemen of New York city have agreed with the commission to privately contribute $122,500 ($10,000 having been paid), provided that the legislatures of New York and New Jersey would this winter appropriate enough money to enable the commission to acquire all the property between Fort Lee and Huylers Landing, opposite Yonkers, and to construct a driveway at the foot of the Palisades. Options have been secured by the commission upon this remaining property on exceedingly reasonable terms, the vertical face of the cliffs to be donated to the commission outright. As will be seen from the above report, the commission has expended no money whatever for office rent, clerk hire or incidental or personal expenses.

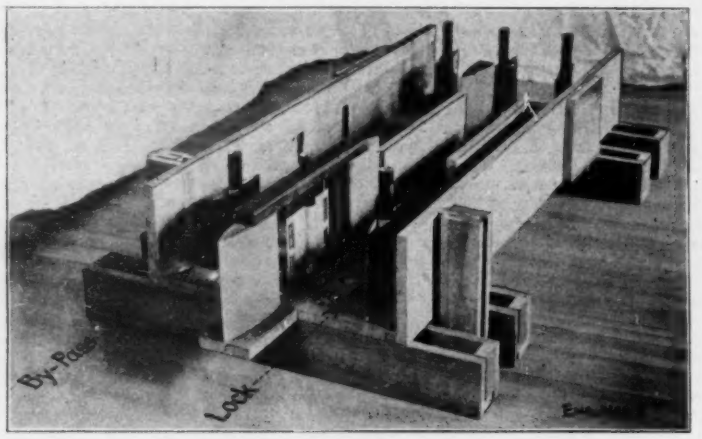
THE LICENSING OF ARCHITECTS is under consideration in New York and a convention of the architects of the state has been called, to be held at the Architectural League Rooms in New York city, Jan. 19. A special committee of the League will report upon a bill which it has drafted for presentation to the legislature. The bill is framed on similar lines to the Illinois law (Eng. News, 2d Vol., 1897, p. 268). A similar bill passed both branches of the New York legislature in 1894, but failed to secure the Governor's signature.

SLOW SAND AND MECHANICAL FILTRATION for Washington, D. C., will be discussed at the Waldorf-Astoria Hotel, in New York city, on Jan. 4, 1901, before members of the Senate Committee on the District of Columbia. It is expected that several district officials and a number of specialists in water purification will be present.

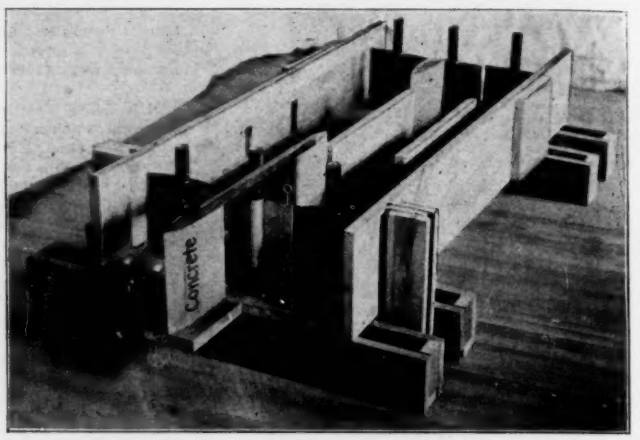
PUBLIC CONTROL OR MUNICIPAL OWNERSHIP of street railways in Illinois are alternatives authorized by a carefully drawn and comprehensive bill submitted to the city council of Chicago on Dec. 17, 1900, by the street railway commission of the council. The plan for municipal ownership is novel in that it limits bonds for street railway construction to a lien on the street railway property, instead of having the lien embrace the whole credit of the city. Mr. Milton J. Foreman is chairman of the aldermanic commission, and Mr. Geo. C. Sikes (who is not a city official) is chairman.

DESIGN FOR CONTROLLING WORKS AT THE HEAD OF THE CHICAGO DRAINAGE CANAL.

In a recent discussion on the relation of the Chicago River to the commerce of the port of Chicago, which took place at a meeting of the Western Society of Engineers and was noted in our issue of Nov. 29, reference was made to the desirability of having controlling works at the head of the canal. In case of accidents due to vessels being caught by the current, it takes 10 to 14 hours to check the current by raising the dam and closing the gates at Lockport, while controlling works at the head of the canal would promptly relieve the emergency. A design for such works has been prepared by Mr. Ossian Guthrie, M. W. Soc. C. E., New York Life Building, Chicago, and is illustrated in the accompanying views taken from a model. Fig. 2 shows the separate parts, and Fig. 1 shows the works with gates open and closed. In brief, he proposes to form a lock the full width of the canal, subdivided by a central wall, the left-hand channel being a by-pass and the right-hand channel a lock. This latter is further divided by a wall serving to protect the gates from being struck by vessels. At each end of the lock are three gates pivoted on vertical shafts passing through the center of each gate. In each end of the gate is a wicket or smaller gate, and by opening one of these, and so reducing the area, the pressure of the water against the other half of the gate will tend to open the gate. In the model only the center gate is shown provided with these wickets, but in practice all would be alike. The following is an abstract of a paper on this subject read by Mr. Guthrie at the meeting of the Western Society of Engineers on Dec. 19:



Lock and By-Pass with All Gates Open.



Lock and By-Pass with All Gates Closed.

FIG. 1.—MODEL OF PROPOSED CONTROLLING WORKS AT THE HEAD OF THE CHICAGO DRAINAGE CANAL. OSSIAN GUTHRIE, DESIGNER.

The plan for controlling works for the drainage channel, to be located near the junction of the channel and the river, has been designed by me for the purpose of maintaining a uniform current which the pilot may learn to navigate so as to avoid accidents to vessels, to afford immediate relief to vessels jammed across the river and held by the current, and to afford prompt aid to vessels stranded upon the tunnels. It is obvious that none of these desirable objects can be attained by any controlling works 28 miles away. One set of controlling works is in existence at Lockport, at the western end of the canal, 34 miles from the mouth of the Chicago river. It has been found, however, that these works are so far away that in case of a vessel becoming jammed across the stream, as has happened during the season just closed, from six to eight hours are consumed in bringing the canal to a level and checking the current so that the vessel may be released. Consequently, there has been a demand from the navigation interests concerned in the improvement of the river for a second set of works, to be located at the eastern or river end, which will stop the current in much quicker time. Several members of the board of trustees of the Sanitary District have favored the construction of such works, and they were recommended by President Boldenweck in his retiring message, read Dec. 5, of this year, as follows:

On several occasions I have pointed out to the board the need of a second set of controlling works, to be located at the Chicago end of the canal, at or near Kedzie Ave. With a proper construction of this sort, it would be possible to shut off the flow into the canal from both connections with the Chicago river, and so to give quick relief to the vessels that occasionally become unmanageable in the current of the South Branch. At present it is necessary, in case of accident to a vessel, to close the Lockport works; but these are so far away that at least eight hours are consumed in stopping the flow. With such works at this end as I believe can be built at a moderate cost and without preventing navigation between the canal and the river, it should not take more than from one to two hours to bring the river to a quiet level. Furthermore, in case of an accident to the Lockport controlling works, or of a serious break along the canal, a second set of works might save incalculable injury at Lockport and Joliet, while it would also be highly useful in case of repairs being needed at any point along the channel. Indeed, it seems to me that it should be a fixed principle in any construction where human life and property are at stake to provide double safeguards. Should an emergency ever occur where the need of a second set of controlling works would be shown at disastrous cost, the board of trustees would never be forgiven by the public, and its members would never cease to reproach themselves for that lack of foresight.

An approximate calculation shows that with suitable controlling works, such as recommended by President Boldenweck, quick relief could be furnished with certainty to vessels caught by the current. At Main St., where one big boat was jammed across the stream, relief could be had in 15 minutes; at the P., Ft. W. & C. Ry. bridge, where the "Amazonas" swung into an immovable position the day after the full current was first allowed through the canal, only about 20 minutes would be required; while at Washington St. tunnel the effect of the closing of the controlling works would be felt within an hour. At this latter point, great relief would also be afforded to the vessels that frequently become stuck on the top of the tunnel, by the temporary rise of water due to the hydraulic ram effect of suddenly closing the works, the rise at this point being calculated to be about 9 ins. At other points along the river it would also be very beneficial to have a temporary increase of a few inches.

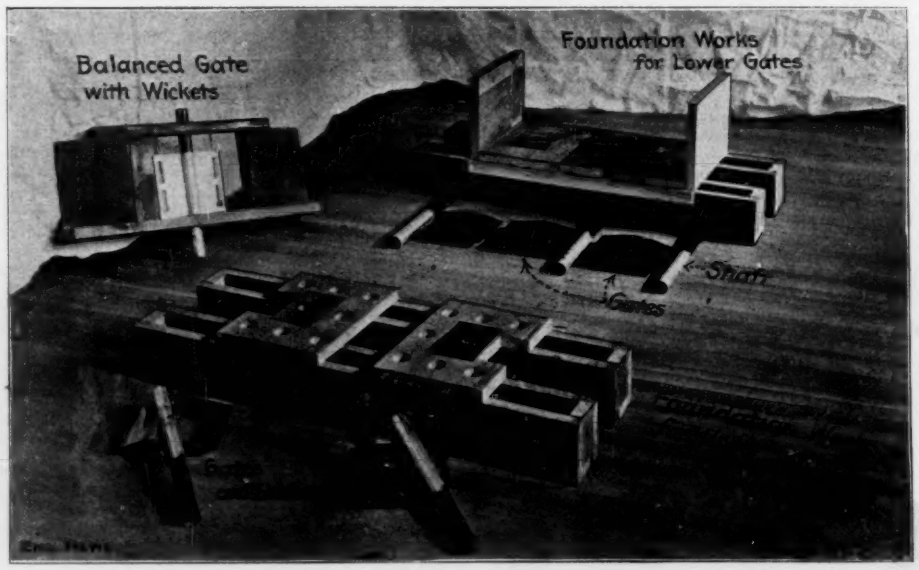


FIG. 2.—VIEW OF PARTS OF MODEL OF PROPOSED CONTROLLING WORKS.

The controlling works proposed should be designed to withstand a head of 16 ft. Under ordinary conditions this would be the utmost that could come against the gates, but if any extraordinary calamity should occur, whereby the lower end of the canal should be opened beyond the control of the Lockport works it would be necessary to hold back the water of the canal east of the Chicago works against a dry channel below. This would be a head of 25½ ft. with the lake at datum. By the plan proposed, this head could be divided between two sets of gates, each sustaining one half, so that the maximum that needs to be considered for each gate is the 16-ft. head.

The Sanitary District law prescribes that the canal after its opening must be a navigable waterway, which necessitates devising some means for passing the largest size vessels at all times through the controlling works. For this purpose the two sets of gates are arranged so as to form a lock, but the sides of the channel are left in their original condition instead of being walled with expensive masonry.

Essentially the device consists of using a number of balanced gates, each swinging on a vertical axis, at each end of the lock. The channel requirements are to allow a flow of 600,000 cu. ft. a minute when the works are open, and to provide for a cross section 183.25 ft. wide on the bottom, a depth of water of 24.5 ft. with the lake at datum, and side slopes of 1 on 2. For the sake of reducing the velocity at this point the work has been planned to have a width of 200 ft. between bulkheads.

It is highly desirable to have the controlling works so located as to shorten to a minimum the distance between the works and a vessel in trouble. To this end it might be desirable, and probably would be economical, to erect a cheaper and smaller set of gates for controlling the flow at Kedzie Ave. and another set for assisting vessels at Robey St. or thereabouts. The structure for relieving navigation should be capable of being operated with the same facility that a bridge is opened or closed. It is also desirable that it should be so made as to provide a uniform flow through the river at all times, thus counteracting the effect of the varying head due to the fluctuations of the lake level. This fluctuation sometimes amounts to 2 or 3 ft. in as many hours. A change of wind usually produces from 12 to 18 ins. in possibly an hour. Consequently there is a variable current in the river which it is hard for navigators to calculate in advance. During the past season the navigation men have made many complaints about the varying flow, and have ascribed it to the raising and lowering of the Lockport bear-trap dam; whereas it was probably due more to the change in the lake level than to any influence at the end of the canal.

By means of two automatic gages, electrically connected with suitable operating machinery at the works, one being located at the mouth of the river and the other at the works, it would be possible to control the flow to any required speed of current.

Each set of gates is three in number, each gate being 66 ft. 8 ins. wide. The central vertical post of each gate will consist of a hollow steel shaft, 3 ft. diameter and 3 to 5 ins. thick and 35 ft. long. The gates will also be made of steel, designed to meet the strains due to the pressure of the water. They may be either constructed in two parts and bolted together around the shaft after the latter is set in place or they may be placed in position first and the shafts then stepped through them.

Each set of gates will rest on a pontoon that can be built anywhere and floated to the location. This pontoon should be 240 ft. long, 18 ft. wide and 8 ft. deep, with bulkheads about 20 ft. from each end and high enough to be level with the top of the channel banks when the pontoon is sunk to the level of the bed. The pontoon will be constructed of two members 6 ft. wide and 8 ft. deep, placed 6 ft. apart and decked across from out to out, with strong timbers bolted through from top to bottom. Through this deck, and extending nearly to the bottom, heavy steel sockets or wells will be inserted and firmly bolted to the structure with through bolts, so as to include the whole mass of concrete with which the entire pontoon will be filled.

To place the pontoons, trenches will be dredged for each and carefully levelled. At the lower side of each trench a row of supporting piles will be driven to hold the pontoon in position before it is sunk. This row will also add to the resisting capacity of the structure against the head of water when the works are closed. After the pontoons are sunk the piles will be cut off level with the pontoon deck, or 24½ ft. below datum. Through the center of each trench a row of Wakefield sheet piling will be driven to hardpan or bed-rock, from bank to bank. This sheet piling will be cut off uniformly about 3 ft. below the channel bed, between the bulkheads, so as to leave a space of about 2 ft. between the top of the piling and the deck of the pontoon. At the sides the top of the sheet piling will be level with the top of the channel banks.

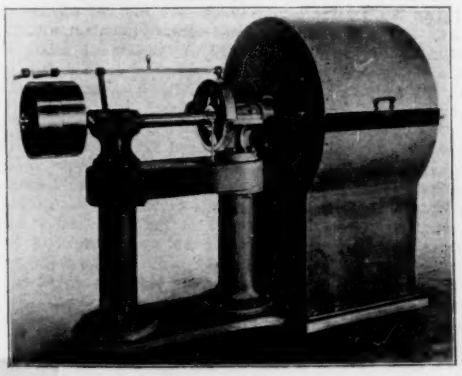


Fig. 2.—View Showing Rattler for Testing Paving Brick, Closed Ready for Operation.

The pontoons having been constructed, supposedly water-tight, they will be launched and floated to position against the supporting piles, when the flow of the canal will be stopped for a few hours and the pontoons sunk by placing in them sufficient concrete to nearly sink them, when the admission of water will be sufficient to cause them to go to the bottom of the trench. Then they are to be filled completely with concrete through openings left in the deck for the purpose, the whole work occupying only a few hours.

As soon as the pontoons are filled with concrete the current in the channel can be resumed, and the work of placing the gates begun. It ought to be possible to complete the whole structure within two or three weeks after the sinking of the pontoons.

It is obvious that there will be need of strongly supporting the tops of the gate shafts. This can be done by a strong strut extending up and down stream from the shaft, and abutting against a stone or concrete pier. The lower end of the shaft, being imbedded in the concrete-filled pontoon, will be firmly secured against moving.

To turn one of the gates it will be necessary only to open one of the smaller or wicket gates, with which each of its two equal leaves will be provided, when the inequality of pressure on the two sides will be sufficient to cause a rotary motion. In this way elaborate machinery can be entirely dispensed with for operating the works. Security from the freezing of water around the central shaft can be provided by making the sleeve at top and bottom about 3 ins. larger than the diameter of the shaft and filling this and the space around the bearing ring with oil, for the full length of the shaft.

Though the gates will be balanced by equal pressure on each side of the central shafts when closed, they will be locked so as to prevent any possible motion. The flow of the water will probably be sufficient to open the gates and to close them after a slight starting power has been applied. For the starting power a small electric motor, taking its current from the nearest available source, can be used.

It is obvious that the number of gates can be varied so as to meet the best requirements of economy and efficiency. If three gates are used in each set, two will provide for the requirements of navigation and the third can be used to regulate the current. The clear opening available for vessels, by the three-gate plan, would be about 60 ft., or 10 ft. more than the width of the broadest vessels now afloat on the great lakes. The distance between the two sets of gates should be enough to allow clearance to the longest craft that may ever use the canal.

According to careful estimates that have been made by responsible contractors, the cost of such a controlling works as has been outlined should not be more than $200,000. On account of the simplicity of construction no heavy contingent expenses will be necessary in the construction; not a dollar will have to be spent for coffer-dam construction and maintenance or for pumping. The whole structure will be as little liable to injury or the necessity of repairs as the best constructed bridge, and built as described it should be as permanent as may be desired.

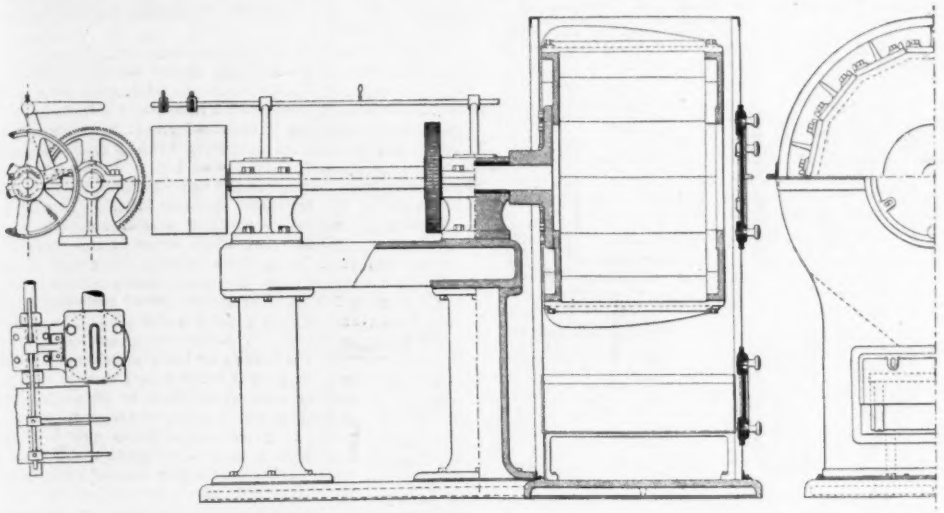


FIG. 1.—SECTIONAL ELEVATION AND DETAILS OF RATTLER FOR TESTING PAVING BRICK; PURDUE UNIVERSITY.

NEW IMPACT TESTING MACHINE AND BRICK TESTING MACHINE, PURDUE UNIVERSITY *

By William Kendrick Hatt† and W. P. Turner.‡

RATTLER FOR TESTING PAVING BRICK.

An ordinary foundry rattler of size approximating the standard, is commonly used by engineers in conducting rattler tests for paving brick. Such a machine is hardly in keeping with the more perfect apparatus of a testing laboratory. The requirements demand a rattler which, in addition to presenting a good appearance, shall be easily operated, and for the protection of the other testing apparatus, keep the air of the laboratory free from dust. Such a rattler was designed and built in the machine shop of Purdue University, as shown in Fig. 1, and has a place in the Laboratory for Testing Materials. Subsequently a similar machine was constructed for Columbia University, New York; and advantage was taken of the experience with the first rattler to improve certain details. The views shown in Figs. 2 and 3 are of the improved machine. Fig. 1 shows a section and a half end view. The drum is 20 ins. long, and 28 ins. in diameter. There are 14 cast-iron staves, with ¼-in. openings between. The staves shown in Fig. 1 proved too weak to withstand the service, and were rebuilt. Instead of one rib along the center line of the stave, heavy ribs were placed on the edges. There is excessive wear on the staves, which should be built of steel.

The drum is overhung and is supported by bolting one head to a 14-in. cast-iron face plate, which is secured to a 3½-in. shaft. The other head of the drum is thus free and an opening is provided therein for charging or removing the bricks, which process demands, in case of the ordinary rattler, the removal of a stave. The arrangement thus saves time, and allows the repeated weighing during the test, which are necessary in scientific experiments. In case small shot are used as a filling material, the opening in the head is closed by a plate.

The main shaft is supported on two bearings, resting on a heavy cast-iron box yoke, which is supported on two cast-iron columns joined together by a heavy cast-iron base. The main shaft is driven from a parallel shaft by a gearing in the ratio of 4 to 1. On this parallel shaft are tight and loose pulleys, 12 ins. in diameter carrying a 4-in. belt.

As a protection from dust the drum is closely surrounded by a sheet-iron casing with felt-packed joints. This casing is made in two sections, the lower part resting on the cast-iron base of the machine; the upper part is hinged to the lower and can be turned back when it is necessary to inspect or repair the barrel. A circular opening in the case corresponds to the opening in the head of the drum; and a plate is quickly clamped by hand clamps to cover the opening in the case. A door in the lower part of the case provides for the withdrawal of a pan in which the dust from the rattler is collected. Fig. 2 shows the machine closed ready for use. Fig. 3 shows the machine with the case opened, and the dust-pan partly withdrawn. In Fig. 2 may be seen a revolution counter on a vertical post. The machine complete weighs about 1,800 lbs., and costs about $200, material and labor included.

AN APPARATUS FOR EXPERIMENTATION IN IMPACT.

A series of tests has been in progress in the laboratory for the past three years to determine the behavior of iron and steel under impact in tension. A temporary wooden apparatus was used for this purpose. The method of testing seemed so promising that a permanent apparatus has been designed and built of iron, with the mechanical details arranged for ease and accuracy of operation. The apparatus has been in operation for two months, and in the judgment of the writers, is a valuable form of apparatus. It is this machine which is described below.

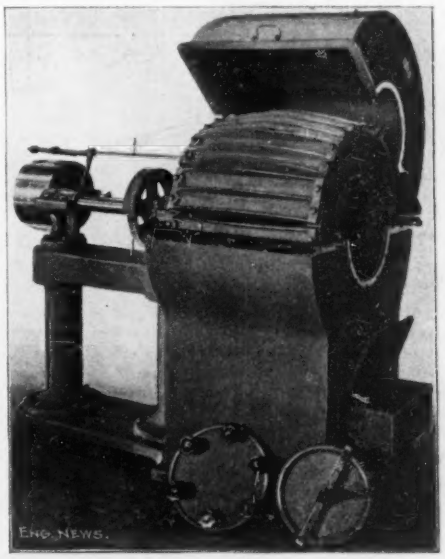


Fig. 3.—View Showing Rattler Opened for Charging.

In the former wooden machine, Fig. 4, the hammer M was hung to a head attached by wedges to the lower end of the speciment B. The specimen and hammer were lifted by an attachment to a similar head C, fixed by wedges to the upper end of the specimen. On release and consequent descent, the upper head caught on a bridge

*Translated from a paper presented at the International Congress of Methods of Testing Materials of Construction, Paris Exposition, 1900.

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‡Instructor in Machine Shop, Purdue University, Lafayette, Ind.

P between the two uprights. The deformation and the work of deformation were obtained from the velocity displacement curve recorded on the surface of a revolving drum D by a pencil attachment to the hammer M. This temporary apparatus was used to bring about rupture in one blow of an 800-lb. or 1,200-lb. hammer falling from a height of from 6 to 12 ft. on iron and steel wires, from ⅛-in. to ⅛-in. in diameter, and of lengths up to 10 ft. Some reference to this work has been made elsewhere.*

The present permanent apparatus is similar to the temporary apparatus in principle. The better adjustment for short specimens is brought about by dispensing with the bridge and constructing the head so as to span the distance between uprights, as shown in Fig. 5. The head or yoke is stopped directly by the columns. The details

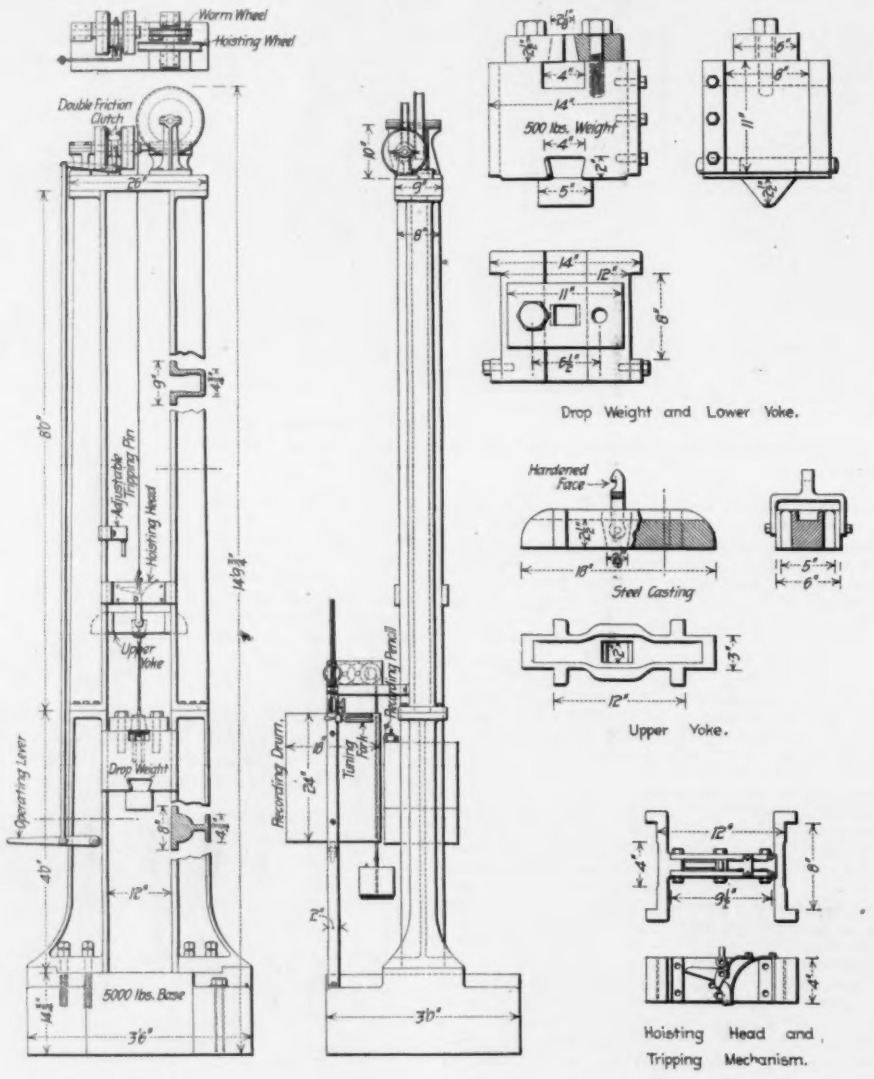


FIG. 7.—DETAILS OF IMPACT TESTING APPARATUS, PURDUE UNIVERSITY, LAFAYETTE, IND.

of attachment of weight, recording apparatus, hoisting mechanism, etc., were made as perfect as possible. The parts were designed to rupture in one blow a test specimen of soft steel ½-in. in diameter, and 8 ins. between heads. Ordinarily the blow delivered is about 3,500 ft.-lbs. Tests in compression are provided for.

Some discussion of the stiffness of the various parts of the apparatus, and some estimate of the amount of energy used in the deformation of the apparatus will follow the description. For the present the paper takes up the principle of action of the apparatus, and the description of the details.

PRINCIPLE OF ACTION.—Given two masses M_1 M_2 joined by a rod S, Fig. 6: The system falls from a height h. M_1 has impact against a mass M_2, and deformation with rupture, of S results.

Let W_1, W_2, be the weights of the masses M_1 M_2, respectively.

Let C_1, C_2, be the initial velocities of the masses M_1 M_2, respectively.

Let V_1, V_2, be the final velocities of the masses M_1 M_2, respectively.

Let $\int Tdl$ denote the work of deformation of entire system.

Let l_1, l_2 denote deflection of M_1 and elongation of S, respectively. Then applying the equation of work and energy between the time when M_1 has contact with M_2, and the time when the rod ruptures, we have neglecting weight of S:

$$W_1 l_1 + W_2 (l_1 + l_2) = \int Tdl - \tfrac{1}{2} (M_1 + M_2) C_1^2 + \tfrac{1}{2} M_1 V_1^2 + M_2 V_2^2.$$

In the apparatus used $W_1 l_1$ will be quite small compared to $W_2 (l_1 + l_2)$, and may be neglected. The quantity $\frac{1}{2} M_1 V_1^2$ may also be neglected.

Thus

$$\int Tdl = W_2 (l_1 + l_2) + \tfrac{1}{2} (M_1 + M_2) C_1^2 - \tfrac{1}{2} M_2 V_2^2.$$

The method of testing involves the measurement of l_1 plus l_2 and of C_1 and V_2 from the record on the drum of a pencil attached to the hammer; l_1 plus l_2 are measured directly from this record while the velocities C_1 and V_2 are calculated by constructing a triangle of velocities, the hypothenuse of which is the record of the pencil on the drum, and the other two sides are, respectively, the velocity C_1 or V_2, and the velocity of the circumference of the drum, which is known from the record of a vibrating tuning fork held in contact with the drum during the test.

Thus the work of deformation equals the product of the weight of the hammer by the total deformation of the specimen plus the difference between the kinetic energy of the system before and after impact. To obtain the work of deformation of the test piece including the elastic energy of the part outside the neck the quantity $\int Tdl$ must be corrected for deformation of the machine parts, such as the head or yoke, the heads of the specimen, the columns, etc. The yokes are calibrated by means of a testing machine of the ordinary type; and thus the elastic energy absorbed in the yokes corresponding to any given deflection is known. This deflection having been determined under the conditions of impact correction can readily be made.

DESCRIPTION OF THE MACHINE.—Fig. 7 shows the machine complete as it stands in the testing laboratory. Its base is partly sunken below the level of the floor, and rests upon a brick foundation 5 ft. × 5 ft. × 6 ft. deep, laid in natural cement on undisturbed gravel foundation; the upper four courses are of paving brick laid in Portland cement. A ¾-in. bearing bed of 1 to 1 Portland cement mortar is disposed between the bed-plate and the pier. By reference to the assembled drawing, Fig. 7, it will be seen that the machine consists of a solid cast-iron base, weighing about 5,000 lbs., which supports two cast columns about 12 ft. high, these columns being tied at the top with a cast box yoke. Details of the base and columns are shown in Fig. 7. The inner face of each column is planed flat, and acts as a guide for the hammer. Each column comprehends two parts which are bolted and dowelled together, the lower part being a solid casting, 550 lbs., while the upper part is cored to a U-shaped section, 525 lbs. A general view of machine is shown in Fig. 5B. The upper yoke to which the specimen is fastened is shown in Fig. 7. It consists of a heavy steel casting* (65 lbs.) with its ends planed to fit inside the U-shaped column or guide. In the center is a tapered rectangular opening into which the specimen is fastened by means of wedges. Provision can be made for fastening by using threads and nuts on the ends of the specimen. The lower yoke by which the hammer is hung to the specimen is also a steel casting, with tapered rectangular opening. This is bolted to the hammer, Fig. 7, with two heavy bolts which are screwed up to an initial tension.

The hammer is a casting weighing 515½ lbs., nicely fitted to the guides with little play, and at the same time little friction. A dovetailed slot is planed in the bottom, into which may be keyed heads of different shapes for making compressional or transverse tests.

For raising or lowering the hammer the hoisting mechanism shown in Figs. 7 and 9 is used. It is so designed that the weight may be raised, lowered or stopped in any place desired. This is accomplished by a worm wheel keyed to the shaft, which carries the hoisting drum. On the worm shaft are two friction pulleys running in opposite directions. These friction pulleys are operated by a rocker arm connected to a vertical operating shaft, passing along one column. Springs hold this shaft and the friction pulley in mid position, the lever being pulled down or up when it is desired to raise or lower the hammer.

For releasing the specimen and hammer from any given height of drop, a special tripping head is used, shown in Figs. 7 and 9. This head is hooked to the wire-hoisting rope, and slides freely between the guides. A bale, suspending the upper head to which the specimen is fixed, carries a hook maintained by a spring to bear on a square pin in the tripping head. When thus engaged, the system may be hoisted until the head comes in contact with the stop which has been adjusted at the desired height on the guides, whereupon the stop will press down the trigger in the tripping head, disengaging the hook and releasing the system.

The circumstances of each impact are recorded upon a revolving drum by a pencil attached to the hammer. The drum is of rolled sheet metal, supported vertically in an adjustable frame, which may be raised or lowered to the proper height for recording tests of long or short specimens. Ball bearings are used on the lower end of the drum shaft to reduce friction as much as possible. A 50-lb. weight attached to clock work furnishes the power for revolving the drum. When used the weight is wound to a certain height and as it falls the speed of the drum is accelerated until the weight strikes a stop. The speed of the drum remains practically constant during the impact. The speed of the drum may thus be varied and nicely regulated by varying the height to which the clock weight is wound.

The exact speed of the drum at the time of the drop is known by allowing a tuning fork with a fine brass point attached to one arm to record its vibration on a piece of metallic paper attached to the upper part of the drum.

The recording pencil is operated by an electro-magnet, so arranged that as the hammer falls electric contact is made after the pencil point has passed below the upper edge of the drum. The magnet then draws up an armature, and presses the pencil against the drum.

When it is desired to make tests on long specimens, the upper yoke must be blocked up. This may be accomplished by setting solid rectangular pieces on the lower sections of the columns, thus filling the lower part of the

*See preliminary report of committee on present state of knowledge of impact tests. Bulletin No. 5 American Section of International Society for Testing Materials.

*This upper head reveals no trace of rupture or cracking after use in breaking 120 test pieces.

U-shaped columns. These pieces are held in place by bolts.

The machine is easily operated and tests may be quickly made. In making a test the operator first fastens the specimen between the hammer and the upper yoke. If wedges are used they are forced into the lower yoke with a pinch-bar, and driven into the upper head with a sledge.

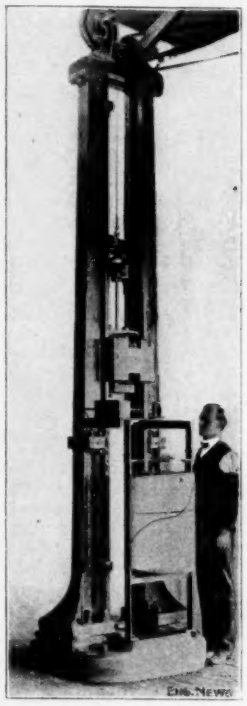


Fig. 8.—View of Impact Testing Apparatus, Purdue University.

By this procedure any noticeable slip of the wedges, with reference to the yoke and any slip of the specimen in the wedges is prevented. The tripping head is then lowered and hooked on to the yoke. A datum line representing the original length of specimen is drawn on the drum. The system is then raised nearly to the point release at the desired height. The clock weight is then wound to the proper height to give the desired speed to the drum. At the instant the clock weight reaches its stop the system is raised to find release. At the same time the tuning fork is brought against the drum to trace its record.

To absorb the shock of the hammer upon the base after rupture of the specimen, blocks of pine wood are placed thereon. The operation of placing the specimen in place and bringing about rupture is accomplished in 6 mins. The time of rupture for the specimens used is 1-100 sec.

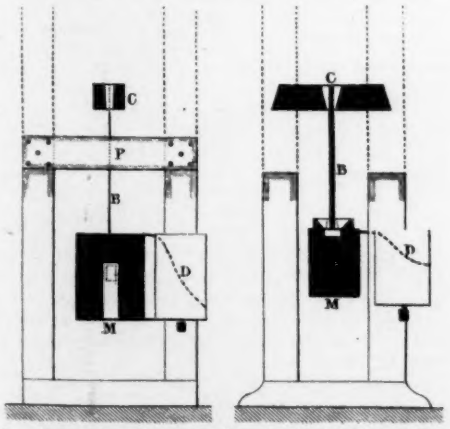
ESTIMATE OF DEFORMATION OF PARTS OF MACHINE.—Referring to equation

$$\int Tdl = W_2 (l_1 + l_8) + \tfrac{1}{2} (M_1 + M_2) C_1^2 - \tfrac{1}{2} M_2 V_2^2,$$

the expression $\int Tdl$ includes the work of deformation of the specimen, (a) of the shackles or wedges; (b) of the heads of specimen; (c) of the yoke M_1; and (d) M_2; (e) of the columns of the machine; (f) work of friction of guides.

(a) and (b). It has been already stated that it was possible to drive the wedges so firmly that no further movement was noticeable.

(c) Before the upper and lower yokes were placed in the machine, they were loaded on an Olsen testing machine with successive loads up to 30,000 lbs., and deflections noted. The yokes were supported at ends, and the load was applied through a specimen attached to the yoke by wedges in the manner of use in impact tests. The deflection of the heads were measured at the center of the supported length, as follows:



Figs. 4 and 5.—Diagrams Showing Original and Final Forms of Impact Testing Apparatus.

	Upper Yoke.					
	—Reading, in ins.—			—Reading, in ins.—		
Load, ins.	Right.	Left.	Aver-age.	Right.	Left.	Aver-age.
200	2.748	2.744	46	2.747	2.747	44
10,000	44	41	42	43	38	41
20,000	41	36	39	39	36	37
30,000	38	35	36	38	35	38
200	..	..	..	47	43	45

The average deflection for these two tests was:

Load.	Deflection.	Load.	Deflection.
200 lbs.	.....	20,000 lbs.	0.0070-in.
10,000 "	0.0035-in.	30,000 "	0.0080-in.

In like manner for the lower yoke:

Load,	Right.	Left.	Average.	Deflect'n ins.
200	1 455	1.460	57	..
10,000	62	65	63	0.006
20,000	74	67	70	0.013
30,000	73	82	78	0.021
200	58	66	62	0.005
30,000	75	85	80	0.023

These deflections include the deformation of both lower yoke and fastening bolts.

We have thus a means of computing the energy stored up in these yokes for any given deflection which may occur under conditions of impact when these yokes are in use. In case of M_2 a scriber was firmly fixed to the hammer so that a record would be made on the lower yoke indicating any deflection of the bolts or the yoke. For a blow of 2,000 ft. lbs. of energy no deflection could be detected.

No means of measuring the deflection of the upper head has been devised as yet. It is, however, a reasonable assumption that the relative proportion of work stored up in the two yokes and the specimen remains nearly the same, whether the specimen be ruptured slowly or quickly. The assumption is favored by the fact that the "total resilience"* of soft steel appears to be nearly the same for impact and for slow testing. The inertia of the upper yoke would diminish the work absorbed by the upper head.

If the assumption be granted then, for a nickle steel test bar ½-in. in diameter and 8 ins. between shoulders, whose maximum load in slow test is about 20,000 lbs.

$$\text{The upper yoke absorbs } \frac{20{,}000}{2} \times \frac{0.007}{12} = 5.8 \text{ ft.-lbs.}$$

$$\text{The lower yoke absorbs } \frac{20{,}000}{2} \times \frac{0.001}{12} = 0.8 \text{ ft.-lbs.}$$

As the specimen absorbs nearly 2,000 ft. lbs. the error on this basis is very small.

(e) The energy represented by ½ $M_1 C_1^2$, that is, the kinetic energy of the upper yoke is almost entirely lost in impact against the columns of the machine. For instance, a free fall of the upper yoke from 6½ ft. gave a rebound of only 8 ins. Therefore, for M_1, in the formula, we may put 1-10 M_1.

(f) The friction of the guides during the deformation of the specimen may be estimated by an observation of the retardation of the free fall of the hammer. In nine determinations of the velocity due to a fall of 5½ ft., the average was 18.26 ft. per second, with a range of from 18.44 to 18.02 ft. per second. The theoretical velocity is 18.83 ft. per second. As the moving mass weighs 550 lbs., this retardation corresponds to a constant friction of about 48 lbs. The work of friction during the deformation of the specimen is 48 × 2-12 = 8 ft. lbs.

The method of drawing tangents to the curves on the drum record involves a compensating error. The possibilities of error have not yet been determined. The range of values cited above are, however, an indication.

It is believed that when the wedges are driven tightly, and the apparatus in order, that in the 2,000 ft. lbs. of energy nominally absorbed by the specimen, fully 95% is chargeable to the specimen. Part of this is, of course, represented by an elastic stretch of the rod, which will result in a recoil if the rod is not broken. In a few cases the specimen was stretched to a neck by the blow without, however, rupture taking place. In one case the recoil was ¼-in., representing for the 850-lb. masses lifted, an elastic energy of 12 ft. lbs. In still another case the recoil was ⅛-in., representing an elastic energy of 18 ft. lbs. Of this latter amount, if we reason from the elastic energy in a bar at point of necking in a slow test, there is to be charged to the specimen the amount of 7 ft. lbs. We have then 11 ft. lbs. to charge to the yokes and heads.

During a test of a piece of steel under the action of twelve blows, each from a height of 1 ft., opportunity was given to bring about release while the pencil was in contact with the drum. The curve of the release was to the eye perfectly smooth, showing a very regular action of the release. In a test of this nature in which a number of blows of equal intensity are used, it is interesting to note the hardening effect of the deformation of the specimen. Thus the recoil of the hammer from the first blow was 0.3-in.; from the sixth, 0.5-in.; from the tenth, 0.8-in.

The character of the record obtained on the drum is shown in Fig. 10 by a tracing of a record of a number of steel bars arranged to intersect a common base line. The speed of the drum is variable. A specimen log of impact tests of soft steel at different temperatures is given in Table I.

The machine is at present being used in an investigation of the effect of temperature on the resilience of metals under shock. Nickel steel, soft steel, wrought iron and cast iron are being used for this purpose and the range of temperature at present attained is from — 100° F. to + 400° F. The bars are surrounded with a tube containing the freezing mixture, and broken by a single blow.

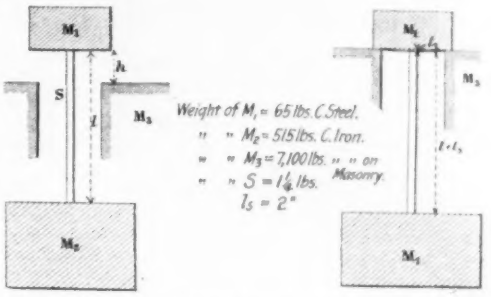


Fig. 6.—Diagram Illustrating Principle of Action of Impact Testing Apparatus.

Difficulty is experienced in bringing about rupture of the bars at the center in experiments at low temperature, even when the wedges and heads of bar are frozen. An unturned bar 12 ins. long was frozen with ice and salt for 4 ins. at the middle, while the ends were left exposed. The bar was broken with one blow. Both ends near the wedges were drawn down to a neck, while the frozen middle center of the length preserved its original diameter and length. Indeed, the elastic limit seemed not to have been exceeded along the frozen length; for the scale was intact and the surface of the bar bright after impact. For this reason the bars may break at the ends. The cause may, however, only partly be due to difference of temperature of the middle of the bar and the ends; for there seems to be a tendency toward localization of deformation of bars at ends even at ordinary temperatures. Shock tests on concrete blocks 6 × 6 × 6 ins. are also in progress.

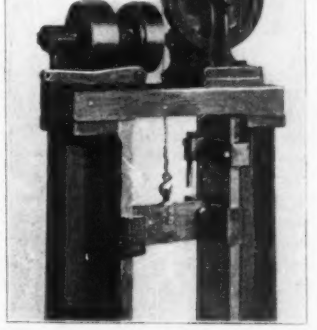


Fig. 9.—View of Mechanism for Raising the Hammer, Impact Testing Apparatus.

*The expression "total resilience" is used for "the total work of deformation to rupture."

TABLE I.—Specimen Log of Impact Tension Tests Made at Purdue University on Mild Steel with One Blow from Impact Testing Machine.

Labo-ratory No.	Materi-al.	Diam-eter, ins.	Weight of hammer, lbs.	Height of fall.	Initial velocity, ft. per sec.	Final velocity, ft. per sec.	Elonga-tion in 8 ins., ins.	Diam-eter at fracture, ins.	Resili-ence in ft.-lbs. per cu. in.	Temp. of mix-ture, degs. F.	Location fracture, from top of bar, ins.
11	4—2	0.511	515½	5 ft. 6 ins.	18.30	8.33	2.02	0.292	1,329	+ 70	+ 1
44	0—5	0.508	515½	5 " 1 "	18.05	13.38	0.68	0.310	743	− 100*	1
30	7—3	0.525	515½	5 " 6 "	18.25	10.72	2.10	0.297	1,080	− 4	8
17	6—3	0.505	515½	5 " 6 "	18.41	7.68	2.10	0.291	1,453	+ 32	1
38	8—3	0.500	515½	5 " 0 "	18.10	8.13	2.00	0.300	1,387	+ 212	4
41	8—6	0.520	515½		17.31	9.84	1.86	0.305	1,000	+ 400	1
37	7—7	0.518	515½	5 " 0 "	17.81	5.91	2.45	0.283	1,405	+ 212	4

*CO_2, alcohol. Remarks.—Test bars were ½-in. in diameter; 8½ ins. between shoulders, with ⅝-in. heads, 4 ins. long.

THE TRANSCONTINENTAL TRIANGULATION AND THE AMERICAN ARC OF THE PARALLEL.

By John E. McGrath.*

In the recently issued volume which contains the details of the astronomical and geodetic operations of the "Transcontinental Triangulation and the American Arc of the Parallel," the United States has contributed to the science of the world, and to American geography in particular, the results of an arc which in magnitude of extension surpasses the achievement of any other government; and whose character for excellence challenges comparison with similar work of the highest merit.

The general appreciation of the value and necessity of surveys, such as the one here referred to, needs no stronger confirmation than is supplied by a reference to the list of membership and reports of the International Geodetic Association. From the roll of European powers the only names missing in the list are Servia, Bulgaria, Montenegro, and Turkey; North America is represented by the United States and Mexico and the new spirit of civilization in the Orient by Japan. The countries wherein geodetic surveys are executed and whose results are reported include, besides the fatherlands of the recognized Great Powers where wealth and high civilization have made exact cartography a necessity, Greece, Roumania, Tunis, and Algiers (which the French, Italian and Spanish geodesists have connected with the continent of Europe), South Africa, India, Java and Sumatra.

This report, which contains the fruits of 27 years' labor, that, under the inspiration and direction of Benjamin Peirce, C. P. Patterson, Julius E. Hilgard, F. M. Thorn, Thos. C. Mendenhall, W. W. Duffield, and Henry S. Pritchett, Superintendents of the U. S. Coast and Geodetic Survey, carried this great work to a successful completion, comes from the hand of Mr. C. A. Schott, Assistant U. S. Coast and Geodetic Survey, who had charge of all the computation involved, from the inception of the work to its close, and whose scientific ability, indefatigable interest and exertions in this great undertaking have been most honorably recognized by his selection to present its matured results to the world. In the text of the report will be found as full meed of credit to the members of the Survey, who, in the field or office, have contributed to the completion of the work, as the space at the author's command would permit.

The successive volumes of the reports of the U. S. Coast and Geodetic Survey, since the inception of the work, have contained a continuous record of its progress, and in their appendices will be found special reports on the instruments employed, the measurements of the bases, the adjustment of the stations in the longitude nets and the results of the precise leveling operations. In the final publication, which treats the Transcontinental Arc as a whole, we find a succinct presentation of every step in the work where conclusions follow from methods that leave practically nothing to acceptance on faith or authority. That a transcript of the record for every base bar laid down, of every direction measured, every micronometric difference of zenith distance at each latitude station and every corresponding detail for the azimuth and difference of longitude determinations should be given, was out of the question in a work of such vast extent, but in declining to swell the report by such intolerable profuseness no reasonable grounds were left for complaint of insufficiency of data. In the treatment of the bases every result is submitted which will enable the reader to follow the various steps from the standardization of the apparatus to the deduction of the probable error of the base from the bar measures. The discussion of the results from the horizontal and vertical angles begins with the abstracts of the angles at each station, and abstracts containing every result at a station are furnished for the astronomical operations. Full explanations of the adjustments, computations, and the various criteria by which the accuracy of the operations have been judged are given and recapitulations of the formulae employed precede their application in the actual reduction of the work.

The measurement of the Transcontinental Triangulation was directed by Congress by the Act of March 3, 1871, providing "for extending the triangulation of the Coast Survey so as to form a geodetic connection between the Atlantic and Pacific coasts of the United States." At that time the geodetic data available for incorporation in the proposed arc comprised the results from the measurement of the Kent Island base, the angles at 13 stations of the triangulation along the Atlantic Coast, 7 azimuths observed in connection with the same triangulation and 19 latitudes. The completed arc depends upon ten bases; horizontal angle observations at 308 primary and 240 secondary stations; an unbroken line of precise levels from Sandy Hook, New Jersey, to Rock Creek, Wyoming, supplemented by vertical angle measures which complete the hypsometric connection from ocean to ocean; 109 latitude stations, 73 azimuth stations, and observed differences of longitudes at 37 stations.

The unit of linear value employed is the meter, and the first representative of this quantity used as a standard was one of the iron bars made for the committee of scientists from different countries that prepared the original units of the metric system. Tralles, the Swiss member of the committee, presented the bar, called "the Committee meter of 1799," to Mr. Hassler, the first Superintendent of the Coast Survey. By him it was given to the American Philosophical Society of Philadelphia; the Society, in turn, placing it at the service of the Coast Survey. In 1889, the United States received from the International Committee of Weights and Measures three platinum-iridium copies of the International prototype meter. Through comparisons of the one selected as the National standard meter with the "Committee" and the "Repsold Steel" meters (the latter being the standard of length for the Olney base of the U. S. Lake Survey which was incorporated in the transcontinental triangulation) the results for the whole work are reduced to the terms of a unit whose relation to the standards of all countries engaged in important geodetic work is known.

The total length of the ten bases is almost 53.5 miles; the St. Albans, 2.4 miles long, is the shortest, and the Yolo, 10.87 miles long, is the longest base. The probable error of the Kent Island base, measured in 1844, is 1-128,000 of its length, or about 0.5-in. to the mile; for the Salt Lake base, measured in 1896, the probable error is 1-1,600,000 of the length, or less than 0.04-in. to the mile. These probable errors do not represent that of the measurement only; they include the uncertainty of the standardization of the apparatus, the error in the correction of the thermometers, the error of the measurement, and the uncertainty in the height of the base. The probable error of the total length of the Salt Lake base from the measurements with the "Eimbeck Duplex apparatus" was 0.10-in., about 1-70-in. per mile; the probable errors due to the outstanding uncertainty in the length of the measuring bars and their temperature corrections were 0.06-in. and 0.16-in., respectively; the probable error of the intermediary through which the apparatus was referred to the National Standard added 0.10-in. to the uncertainty; and the elevation of the base line above sea level being in doubt by about 8 ft., adds 0.17-in. The combined effect of the five terms gives for the base reduced to sea level, the value already quoted.

The measurements at the Holton and St. Albans bases produced results of great interest to the geodesist and engineer. At Holton, Indiana, the Woodward steel bar-in-ice was first used for the measurement of a standard hectometer and kilometer section, and its successful operation proved that it solved the problem of supplying a practicable form of field apparatus from which the varying temperature effects, the chief source of error in all high-class metrology, were eliminated; here also was put to a satisfactory test a modification of Jaderin's method for the use of metallic tapes for geodetic base work. At St. Albans, West Virginia, is the first base of the triangulation measured exclusively with a steel tape. The standardizing section for securing the length of the tape was 100 meters in length, and was determined with a 5-meter bar-in-ice. The length of the base was 2.4 miles, very nearly; the probable error of the base due to uncertainty in length of tape was 0.09-in.; the probable error due to measurement was 0.09-in.; and the quantity due to uncertainty in reduction to sea level was 0.04-in.; resulting probable error for base reduced to sea level is 0.14-in., or about 1-17-in. per mile.

A large variety of instruments was used in the angular work, but it can be broadly stated that in the large figures west of Pike's Peak the 20-in. direction theodolites were employed; that in the region between the Rocky Mountains and the Alleghenies, 12-in. theodolites were used; in the Alleghenies the work was executed with 20-in. theodolites; in the Blue Ridge a 30-in. instrument was used at some of the old stations, and 20-in. theodolites at the recent ones; the work about the east end of the arc was executed with 12 or 24-in. instruments.

At nearly all the stations the method of observing directions in positions and series was employed. The largest error in the closure of a triangle is 6".5, and in 701 triangles for which results are now published there are but 5 instances of a closing error as large as 5". The average error of closure from 357 triangles that show an excess, is 1".07; the average deficiency in 343 triangles that do not fill exactly, is 1".05; one triangle closes exactly. An interesting comparison is afforded by the respective values for "mean error of angle" from the 39th parallel triangulation, and those derived from the larger operations of some foreign countries engaged in similar work. "Mean error of angle," used for a convenient measure of relative accuracy, is the term $a = \sqrt{\frac{[\triangle^2]}{3\,n}}$ where $[\triangle^2]$ represents the sum of the squares of the excess or deficiency in the closure of each triangle, and n is the number of triangles:

Country.	Epoch.	No. of triangles.	Value of a.
Austria-Hungary	1850 to 1888	674	± 0.91
Belgium	1851 " 1873	219	0.89
France and Algiers	1792 " 1880	914	1.29
Great Britain	1787 " 1865	552	1.83
India	1860 " 1880	1,417	1.00
Italy	1863 " 1890	514	0.92
Prussia—A	1847 " 1877	79	0.73
Prussia—B	1832 " 1891	690	0.55
United States (39th parallel)	1846 " 1897	701	0.77

The following table shows in proportional terms the lack of agreement of the adjacent bases as computed through the intervening triangulation:

Base lines.	Approximate distance apart.	Proportional discrepancy.
Kent Island and St. Albans	339 miles.	1/295,000
St. Albans and Holton	205 "	1/181,000
Holton and Olney	148 "	1/61,200
Olney and American Bottom	108 "	1/734,000
American Bottom and Versailles	150 "	1/60,500
Versailles and Salina	260 "	1/25,700
Salina and El Paso	376 "	1/47,000
El Paso and Salt Lake	541 "	1/50,840
Salt Lake and Yolo	668 "	1/52,600

The tremendous range of a triangulation crossing the continent from ocean to ocean developed conditions which necessitated the application of every refinement known to geodesy in the reduction of the operations, while the execution of the field work made it imperative upon the observers to devise means for overcoming difficulties that at times seemed insuperable. From California to Colorado the arc advances in the largest consecutive series of polygons in the world; 20 of the lines are over 100 miles in length and one line of sight, from Mt. Uncompahgre to Mt. Ellen, is 182.7 miles in length. In this section signals of wood or stone, aside from their expense, were inadmissible because of the atmospheric conditions which affect the visibility even of the great mountain tops. On the long lines occurring there, the observations were made upon heliotropes, simple and cheap in construction, requiring only the attention of a class of service that could be secured anywhere, and which shone with the brightness of a first-magnitude star through haze and vapor that often made the mountain tops indefinable.

In the regions between the Mississippi River and the Rocky Mountains, and between the eastern end of the arc and Cincinnati, center-poles of

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various designs were generally employed; but across Illinois, Indiana, and part of Ohio, the character of the country, much of it a gently rolling prairie, heavily wooded or low hilly type, made it necessary to observe at night upon lamps often raised to great heights. In California, at Great Caspar station, a giant redwood tree was cut off about 100 ft. from the ground. About this the tripod for the instrument was constructed, while the scaffold upon which the observer stood was built out from the tree trunk. In Indiana, at Green station, the instrument was mounted on the tripod head 152 ft. above the ground, and the observing lamp was placed about 40 ft. higher. At Still Pond, Maryland, the instrument was used at an elevation of 120 ft., and the observing pole was raised 155 ft. higher. Stayed with wire guys, this pole made a perfectly satisfactory object for observing.

The distribution of the latitudes, longitudes, and azimuths, which form so essential a portion of the operations of every great geodetic survey, was carefully made to insure as far as possible (within a reasonable limit), an elimination from the resulting geodetic positions of the triangulation points of the effect of local deviation of the vertical (station errors), an error which in nearly every case vastly exceeds the uncertainty of astronomic positions due to observation errors and inexactness in the star places. The average error of a latitude determination in this work is ±0″.10, of the difference of longitudes ±0s.011; of an azimuth ±0″.23; but the average value of the "Station Error" is estimated to be 2″.4 for the stations between Cape May and Colorado Springs. In the mountainous regions to the westward it is greater. The correction of these astronomical results for variation of the pole is supposed to be the first instance of the application of that refinement in geodetic work.

The value of the length of the arc of the 39th parallel, which is now completed and available for use in the determination of the earth's figure, is 4,224,010 meters. Its astronomical amplitude is 48° 16′ 47″.83, and the geodetic amplitude is 48° 16′ 39″.57. The probable error of the length value is 26 meters. Computing the value of a similar arc from the elements of Clarke's and Bessel's spheroid, the results differ by 615 meters. Th: high measure of accuracy of the length of the American arc, and the fact that it lies intermediate between corresponding values deduced from the Clarke and Bessel figures, seems to indicate that only very small changes will ever have to be made in its standard geographical positions because of reference of the work to a more truly representative figure of development.

The value of a determination of the earth's figure from an arc of the parallel in these latitudes is not very great, but the United States is now, or will very soon be able to furnish a wealth of material for the determination of the Geoid such as no other nation can claim. Besides the central arc of the parallel, which is equivalent to 37° of a great circle, the Coast and Geodetic Survey has completed the Nantucket Arc of 3° 22′.7; the Pamlico-Chesapeake Arc of 4° 31′.5; the oblique arc from Calais, Me., to New Orleans, La., 22° in extension; and 8° of an oblique arc, northward from San Diego, on the Pacific Coast. The Lake Survey has measured the Lake Superior Arc of meridian 9° 23′.8; and the Lake Erie Arc of the parallel, 8° 47′.5 of a great circle.

In process of rapid execution we have the triangulation along the 98th meridian, which is to extend from our northern to our southern boundary; its extension in Canada and through Mexico being now a matter of consideration by the government of those countries. The figure of the earth computed by Colonel Clarke in 1880 was deduced from arcs whose combined amplitude was 89° of a great circle; a summation of the completed arcs just referred to will show that (including the arc of Peru) there is available on this continent alone an arc amplitude of 96° 12′.6 for the determination of the Geoid. The chief scientific merit of the transcontinental arc measure is its value for the deduction of the earth's figure; but the immediate return for the cost and labor involved in its completion is to be found in the adequacy with which the results satisfy public requirements. The first of these to be noted is the geodetic connection of the triangulations of the Atlantic and Pacific coasts. We are also supplied with a series of cardinal geodetic positions across the continent which will enable the geographer to proceed according to scientific, and not empirical, methods, in co-ordinating the results of the many valuable surveys made in the interior. In 15 states and the District of Columbia is a network of triangulation, which furnishes bases and azimuths of a uniform quality from which the surveys of the states, their boundaries, county and township lines can be extended by the engineer with the ordinary instruments of his profession, the triangulation furnishes him with his base for starting and checks for the control of his work. The line of precise levels which formed part of the program for the geodetic connection has not yet reached its Pacific terminus, and the elevations of the work depend upon vertical angle observations for the section from Pike's Peak to Pt. Arena. The results from the levels of precision have been published for that portion of the line between Sandy Hook, New Jersey, and Denver, Colorado. Between the tide gauge at Sandy Hook, and Kansas City, the results have been adjusted in combination with values from all intersecting level loops, these latter furnishing six independent connections with the tidal planes of the Atlantic Ocean and the Gulf of Mexico; adjusted results are in press which carry the precise levels as far west as Rock Creek, Wyoming. In nearly every branch of his profession the engineer is forced to consider the question of heights, and no scientific man can be more appreciative than he of the importance of an exactly determined system of bench marks, referred to the most invariable plane at our command, such as the precise levels of the transcontinental arc supply.

It is earnestly to be hoped that the possession of data so valuable as that which the General Government has placed at the service of the States, in the case of the transcontinental arc, will lead to the inauguration of surveys on their parts which, expanding with satisfactory accuracy, will enable them to prepare correct maps of their areas, and bring within reach of the individual the opportunity to refer his private bounds to witnesses that lack the ephemeral and arbitrary character of those he has to use now.

The honor of being the first State in the Union to appreciate the importance of a correctly made survey of her territory was Massachusetts; and while her example has been followed with more or less earnestness by other States, the thoroughness of the Massachusetts work has not been equalled by any of them. Still, it is inevitable that the importance of the interests at stake must soon force other States to action; an exactness of method and practice that is considered necessary for the French possessions in northern Africa, the English possessions in India, and the Dutch territories in Java and Sumatra cannot be unnecessarily refined for the United States.

CHANGES IN THE OPERATING ORGANIZATION OF THE ILLINOIS CENTRAL R. R. CO.

Under date of Jan. 1, 1901, Mr. J. A. Harahan, Second Vice-President of the Illinois Central R. R. Co., announces some important changes in the organization of the operating department of that company. As the matter of organization in railway departments is receiving must attention and the system established by the Illinois Central differs considerably from that in common use, we print the circular as follows:

ASSISTANT GENERAL MANAGER.—The assistant general manager shall report to and receive his instructions from the second vice-president; he shall have general charge of all construction work, maintenance of way and structures; he shall have general charge of the train and station service and general supervision of the preparation of time cards, and shall—subject to the approval of the second vice-president—provide rules for the safe and efficient movement of trains. He shall have charge of the equipment of the company; he shall be responsible for the proper discipline of the employes within his jurisdiction, for the safe and economical movement of the passenger and freight traffic, and for the maintenance of the efficiency of the service, and shall make and enforce such necessary regulations as will best secure the safe and economical operation and maintenance of the road. He shall perform such other duties as the second vice-president may assign to him.

GENERAL SUPERINTENDENT OF TRANSPORTATION.—The general superintendent of transportation shall report to and receive his instructions from the assistant general manager. He shall be responsible for the proper movement of passenger and freight traffic, and shall have charge of the distribution of car equipment. It will be his duty to provide car equipment and arrange for the movement of special trains, and see that regular passenger train service is efficiently and satisfactorily conducted. He shall have supervision of the freight train service and making of schedules for the same, subject to the approval of the assistant general manager. He shall see that cars are moved promptly, and that the most economical and efficient service is obtained from cars and locomotives.

He shall call for information and reports from officers and employes as may be required. The ordinary channel of his official communication shall be through the superintendents, excepting that south of the Ohio river his orders shall be given through the assistant general superintendent. In case of emergency, special instructions requiring prompt action can be issued direct to trainmasters. He shall consult freely with the superintendent of machinery as to the temporary assignment and transfer of engines to the different divisions in order to promptly move the traffic, and with the officers of the operating and traffic departments generally as to the requirements of the service. He shall order from contract mines the necessary supply of coal for the company's use, and make the distribution of same. He shall keep a record of the movement of all cars on this company's lines, and an account of all the moneys that may become due for the use of its cars on foreign roads, and the amount due foreign roads for the use of their cars, and shall furnish such information to the comptroller as will enable him to make settlement of these accounts. He shall perform such other duties as may be assigned him by the assistant general manager.

ASSISTANT GENERAL SUPERINTENDENT.—The assistant general superintendent shall have jurisdiction over the railroads south of the Ohio river. He shall discharge all the duties and exercise all the powers of the assistant general manager which are necessary for the proper management of the several parts of the system under his jurisdiction. He shall be responsible to the assistant general manager and report to him, except as to matters pertaining to maintenance of way, as to which he shall report to and receive instructions from the chief engineer. His office shall be at Memphis, Tenn.

ENGINEER OF CONSTRUCTION.—The engineer of construction shall report to and receive his instructions from the assistant general manager; he shall have charge of all construction work not otherwise assigned to the chief engineer, and shall perform such other duties as the assistant general manager or the second vice-president may assign to him.

A CONTRACT FOR 200,000 TONS OF COAL for the French Government is reported as about to be closed by Mr. F. K. McIlwaine, of the American Chamber of Commerce in Paris. This order covers railway supplies chiefly. Mr. McIlwaine says that the coal supply of France is 10,000,000 tons short of the demand, and that the United States should supply a large part of this.

LARGE COAL FIELDS IN RHODESIA, South Africa, are reported as having been discovered by experts sent out by the British South Africa Co. These fields are about 180 miles northwest of Bulawayo, and cover 400 square miles, with seams 5 to 16 ft. thick. The quality of the coal varies greatly; but it is said to be better than the coal now in use in the Transvaal and Cape Colony. It is now proposed to take the Cape-to-Cairo railway through the center of these fields.

THE BEMENT COLLECTION OF MINERALS, collected by Mr. Clarence S. Bement, of Philadelphia, and one of the very finest gathered by any private collector, has passed into the possession of the American Museum of National History, of New York. Mr. Bement commenced collecting 35 years ago, on the principal that only 'the best was good enough." As a result the collection is said to number about 14,000 speciments, including nearly 600 meteorites, gathered from 445 "falls." The collection is valued at $150,000 to $200,000; but the name of the purchaser and donor to the Museum is not made public.

THE BRITISH PACIFIC CABLE, from Vancouver to Queensland, is to cost $1,795,000, according to the contract reported from Vancouver as being made with the Telegraph Construction and Maintenance Co. For this sum the company agrees to lay a cable 7,986 miles long, connecting the following points: Vancouver, B. C., to Fanning Island, 3,561 miles; Fanning Island to Fiji, 2,093 miles; Fiji to Norfolk Island, 961 miles; Norfolk Island to New Zealand, 537 miles; Norfolk Island to Queensland, 834 miles.

ENGINEERING NEWS
AND
AMERICAN RAILWAY JOURNAL.

Entered at the New York Post-Office as Second-Class Matter.
Published every Thursday
at St. Paul Building, 220 Broadway, New York, by
THE ENGINEERING NEWS PUBLISHING COMPANY

GEO. H. FROST, - - - - - - - - - PRESIDENT.
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ALFRED E. KORNFELD, New York,
M. C. ROBBINS, Chicago,
S. B. READ, Boston,
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PUBLICATION OFFICE, 220 BROADWAY, NEW YORK.
CHICAGO OFFICE, 1636 MONADNOCK BLOCK.
BOSTON OFFICE, 170 SUMMER ST.
CLEVELAND OFFICE, OSBORN BUILDING.

FOREIGN AGENTS.

C. MITCHELL & Co, Snow Hill, Holborn Viaduct, LONDON.
RUDOLF MOSSE, BERLIN and HAMBURG, GERMANY.
JOHN F. JONES, & CIE. 31 Rue de Faubourg Montmartre, PARIS.

SUBSCRIPTION RATES: United States, Canada and Mexico, One Year, $5.00; *6 months,* $2.50; *2 months,* $1.00. *To all other countries in the Postal Union: Regular Edition, One Year,* $7.00 (31 *shillings*); *Thin Paper Edition, One Year,* $6.31 (26 *shillings*). *SINGLE COPIES of any number in current year,* 15 *cents.*

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Amid all the controversy between the expansionists and anti-expansionists, there is one proposition upon which decent citizens of both ways of thinking are agreed. That is the proposition that so long as the United States conducts a government in its tropical dependencies, it shall be an honest government—one conducted in the interests of the people. We are not in the Philippines or in Porto Rico for the purpose of gaining fat contracts and rich concessions; and as for Cuba, we are even more bound in honor to an honest course there, by our public declaration to the civilized world that we would administer the government of Cuba as a sacred trust until such time as its people are competent to carry it on for themselves.

The position of the United States in Cuba was admirably stated by Gen. Wm. Ludlow, M. Am. Soc. C. E., when he assumed the office of Governor of Havana. In a letter addressed to the civil authorities he defined the fundamental principle of the new administration as follows:

> The general purpose is to create an effective and economical administration, solely in the public interest, without reference to party or personal considerations or affiliations.

With these thoughts in mind we turn to Havana dispatches of recent date and read the following headlines:

> DADY'S HAVANA CONTRACT.
> The Struggle Over it May be Transferred to Washington, Where Lemuel E. Quigg will go Immediately—Dady will Appeal to Gov. Wood and He Declares that if the Latter will not Give Him His Rights He will Carry the Case to President McKinley, and Perhaps to Congress.

Now, Mr. Michael J. Dady is a high official in the Republican organization in Brooklyn, N. Y., and Mr. Lemuel E. Quigg has been for some years a prominent figure in the New York Republican organization. Why are these men so anxious to secure the Havana contract? For an answer let us turn to the official report of Gen. Ludlow and read:

> A complete scientific and satisfactory system of modern sewers and pavements for the entire urban area and for a population of 500,000 (double the present one) can be executed at a cost of about one-half that involved in the Dady proposition.

No wonder, with a plum of that sort within reach, Mr. Dady is "fighting hard," as the Havana dispatches put it! What ground has he for his fight? It appears, from the papers before us, that during the Spanish occupation of Cuba, Dady submitted to the municipal authorities a project for a system of sewerage and paving. This project covered not more than one-third of the built-up portion of the city, and provided for a population of only 300,000. The unit prices fixed were exorbitant and large opportunities were afforded for increasing quantities to the benefit of the contractor. The total cost was estimated at $12,000,000 to $13,000,000, and the contractor was to be paid in 6% 50-year bonds at 90. Besides this, he was to receive about $165,000 a year for maintaining the paving and sewerage systems.

Perhaps there was some reason for such prices in the old days of the Spanish regime, when officials had to be "seen" at every turn. But one of the declared objects with which the United States went to Cuba was to throw out the old Spanish system, neck and heels. Even if Mr. Dady had secured a contract from the old Spanish Government, therefore, it ought to be canceled by the United States if it was against the interests of the Cuban taxpayers. As a matter of fact, however, Mr. Dady's contract was never closed. His "rights," therefore, are the rights of the promoter who has been turned down. His project, as Gen. Ludlow says, "has been persistently pushed both in Havana and Washington upon the American authorities since they took charge to the present time; but no authority, technical or executive, that has examined the project has been able to favor it in any sense."

As reported in our issue of Nov. 1, contracts for the complete sewerage and paving of Havana are shortly to be let upon the plans prepared under direction of Major Wm. M. Black, Corps of Engineers, U. S. A., Mr. Samuel M. Gray and Mr. D. E. McComb, all members of the American Society of Civil Engineers. We believe bids would have been called for ere this, save that Mr. Dady has been on the ground "fighting hard." The Ayuntiamento at first turned Mr. Dady's claims down; but two days after the vote was taken, Senor Masquera announced that he had "changed his mind" and desired to vote the other way. On Dec 19 the Municipal Council voted to incorporate in the advertisement for bids the appraisement of Dady's "rights" at over $500,000, and awarded him the right of "tanteo" or the privilege of taking the contract at the figure which any other contractor might bid, and also the privilege of bidding without giving any guarantee. On Dec. 24 Mayor Rodriguez vetoed this action of the Municipal Council, pointing out, what will be plain to every reader, that such action practically destroyed competition, since no bona fide contractor would bid against Dady under such conditions. The Civil Governor of Havana also rendered a decision against the legality of the award to Dady on Dec. 27. Apparently the Cubans are developing the ability to conduct municipal governments which will compare well with not a few cities in the United States.

It is sincerely to be hoped, for the honor of the nation, that Mr. Dady's orators at Washington may meet the rebuke they so richly merit from those in authority. Mr. Dady has conferred not a particle of benefit upon the people of Havana. It will be a disgrace to the United States if they are taxed one red cent for his benefit while the Cuban government is in our hands.

Our American Army officers in Cuba have done noble work. In many cases at large personal and financial sacrifice, and at the risk of life and health they have brought order out of chaos, health where formerly disease ran riot, and honest government in place of corruption. Let them then be upheld.

Such disgraceful episodes as the Cuban postal frauds show the result of trying to administer the government of our new possessions on political lines. One experience of that sort should be enough, and it is to be sincerely hoped that politics may have no part whatever in the final settlement of the Havana sewerage contract.

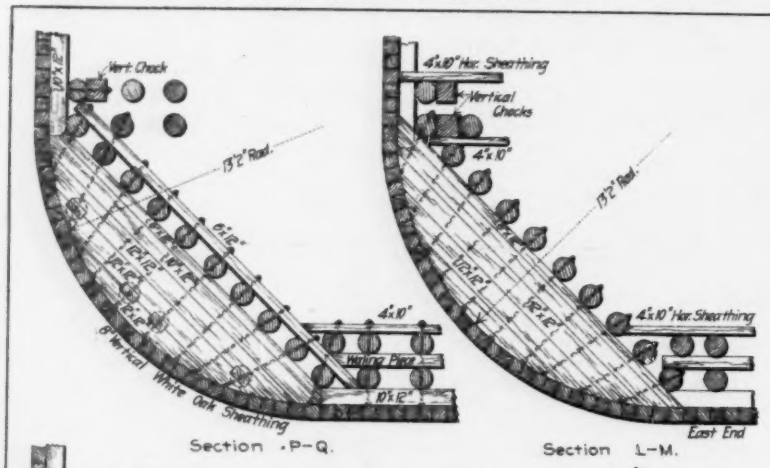
On another page of this issue we reprint an article from our London contemporary, "The Engineer," in which some curious instances are related of defects found in old second-hand girders which it was proposed to use in new construction. The most curious of these is the cast-iron girder which after use in a railway bridge for some 40 years, broke in two with a loud report when a workman attempted to drill a small hole through the center of the web.

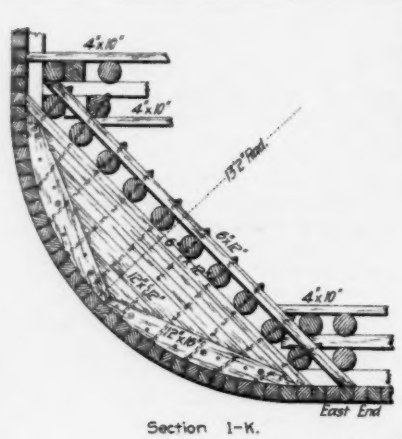
We need hardly say here that cast-iron girders are practically unknown in this country, at least in railway bridge construction, new or old. Their use in new construction has, we believe, been practically abandoned in England, and the instance mentioned above is an excellent illustration of the inherent unreliability which has influenced their abandonment. The condition of internal strain in cast iron is well known, of course, to engineers; but it has generally been believed that this condition of molecular stress would be gradually relieved by the lapse of time. Experiments by Mr. A. E. Outerbridge a few years ago* showed in fact that castings in which internal stress was present gained in strength as time elapsed, and this gain in strength was accelerated by subjecting the casting to mechanical shocks or vibrations. Here, however, is a case, if our contemporary's account is correct, where a large casting retained its condition of internal stress for nearly half a century, carrying meanwhile a heavy load and subjected to the shock and vibration of rapidly moving railway trains.

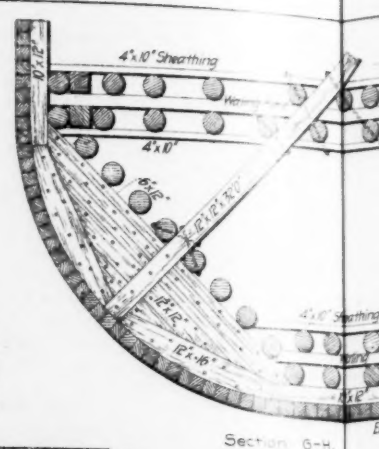
In the face of these facts, the surprising thing to foreign engineers is that American railways should use cast-iron wheels so successfully in all classes of freight and passenger service. Perhaps the greatest reason for the success of the cast-iron car wheel is the thorough annealing which it receives. Next to this is the special quality of iron which is used in its composition, the great care taken to ensure good results, and the elaborate system of inspection and tests which has been evolved in the course of years. It is a fact that the cheap freight transportation which is effected on American railways is due in no inconsiderable degree to their successful use of cast-iron wheels. These wheels cost a cent or less per pound, and when worn out are worth a large part of their original cost to be melted over to make new wheels. The steel-tired wheel costs several times as much when new as the cast-iron wheel, and has small value as scrap when worn out.

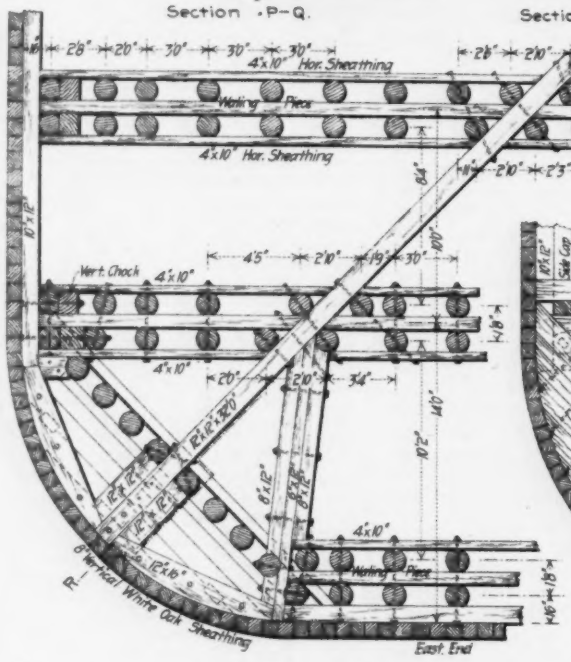
The stories which our contemporary tells of defects in old wrought-iron girders are also interesting. They remind one of the hot controversy as to the comparative merits of pin-connected vs. riveted structures which was waged a quarter-century ago, about the time that these remarkable wrought-iron girders were built. It will hardly be disputed that such riveted work as "The Engineer" describes operated to prejudice engineers against riveted work in general, and goes far to justify the opposition to riveted structures which was current among American engineers a generation ago. The improvement in riveting machinery and shop methods has greatly changed conditions, however, on this side of the ocean; and more intelligent design has largely eliminated the wear of rivets and rivet holes and consequent deterioration of the structure. Riveted work which is so heavily loaded that the rivets move in the plates, will wear in the manner which our contemporary describes; but competent engineers aim to give riveted work a margin of strength sufficient to prevent this, and they provide the necessary inspection to see that the shop methods and workmanship are such as to ensure good results.

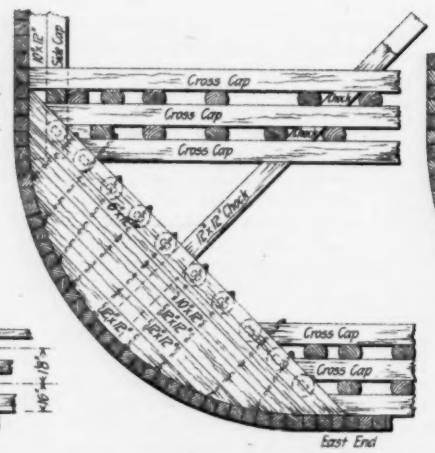
*Eng. News, Feb. 27, 1896.

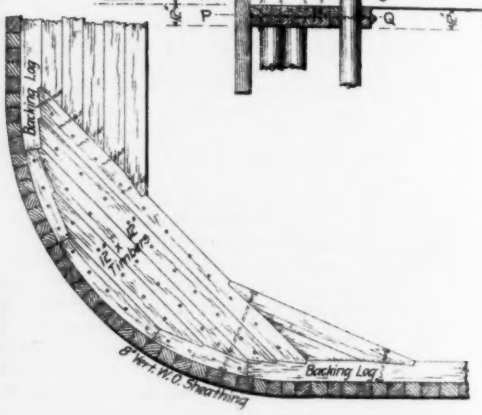












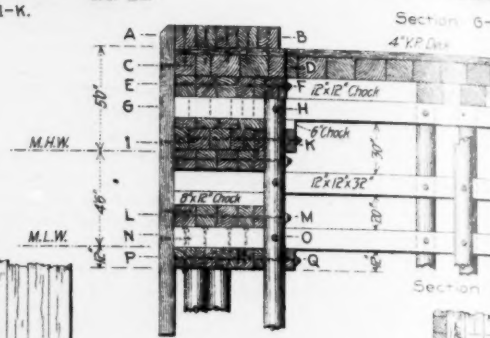
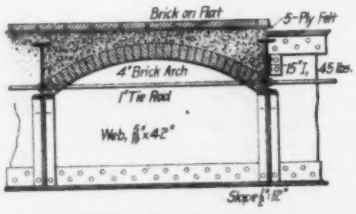


FIG. 10. DETAILS OF CORNER FRAMING AT ENDS OF PIERS.



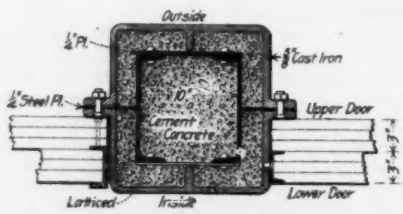


FIG. 21. WALL COLUMN FOR PIER SHEDS.

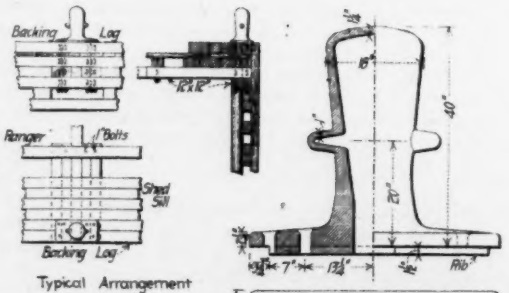
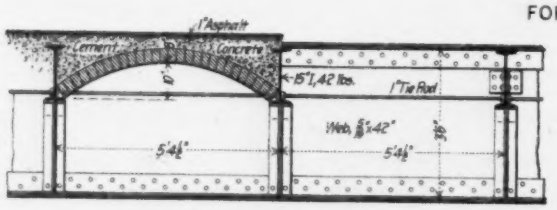


FIG. 17. DETAILS OF MOORING POSTS AND FASTENINGS.

FIG. 9. DETAILS OF PILE REENFORCED BY LAGGING TIMBERS.



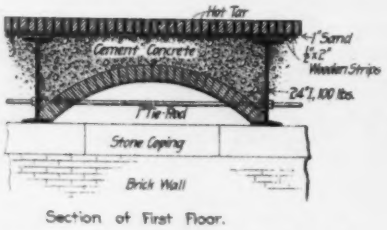


FIG. 5. SECTIONS SHOWING DETAILS OF FLOOR CONSTRUCTION IN BULKHEAD SHED.

FIG. 6. SECTION OF INTERIOR COLUMN FOR BULKHEAD SHED.

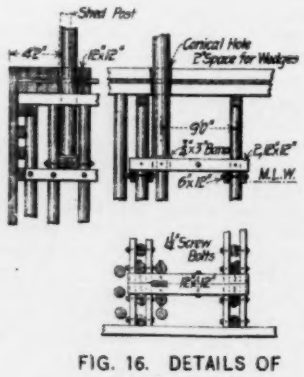


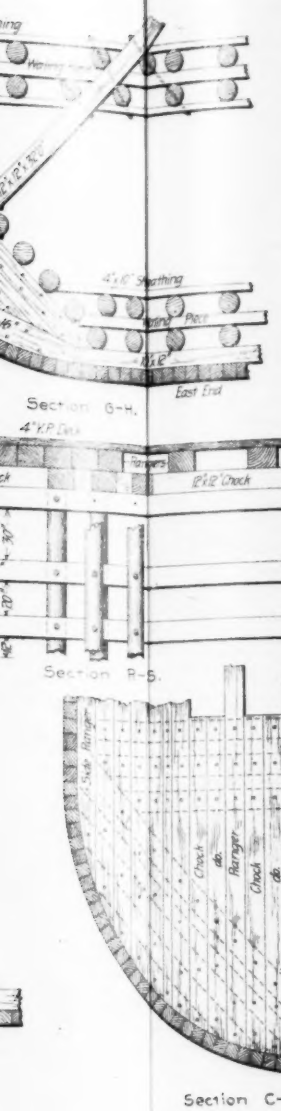
FIG. 16. DETAILS OF DERRICK MAST FOUNDATION.

THE NORTH GERMAN LLOYD STEA

W. F. Whittemore, Assoc. M. Am

Geo. S. Greene, Jr., M. Am. S

R. P. & J. H. Staats, Nev



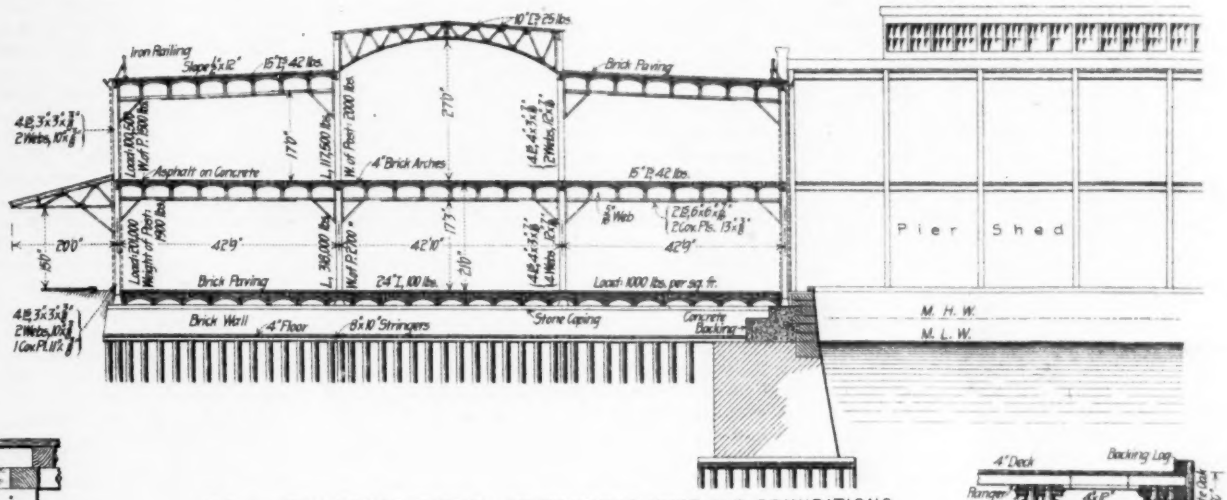


FIG. 4. TRANSVERSE SECTION OF BULKHEAD SHED AND FOUNDATIONS.

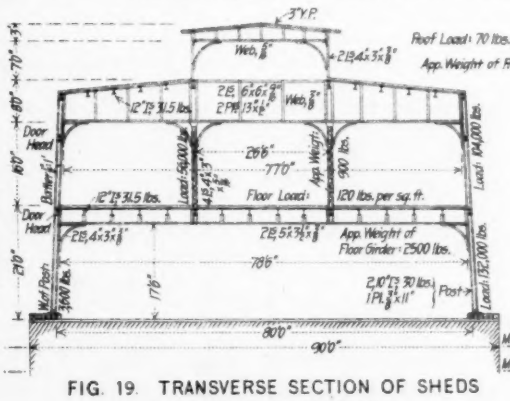


FIG. 19. TRANSVERSE SECTION OF SHEDS FOR PIERS NOS. 2 AND 3.

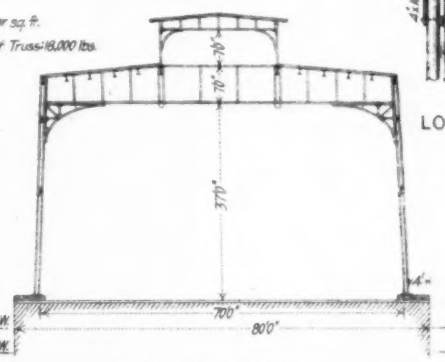


FIG 20. TRANSVERSE SECTION OF SHED FOR PIER NO. 1.

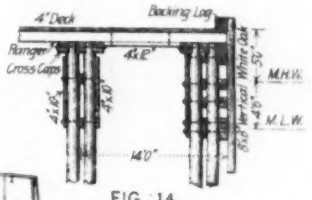


FIG. 14. LONGITUDINAL SECTION NEAR OUTER END OF PIER SHOWING DOUBLE PILES AND BRACING.

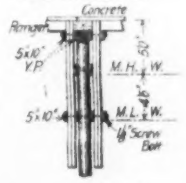


FIG. 15. LONGITUDINAL SECTION SHOWING PILING AND BRACING UNDER FIRE WALLS.

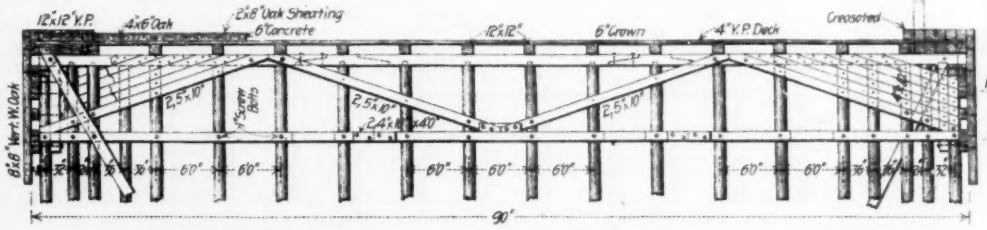


FIG. 13. STANDARD TRANSVERSE BRACING FOR PILES, SPACED 6 FT. ON CENTERS.

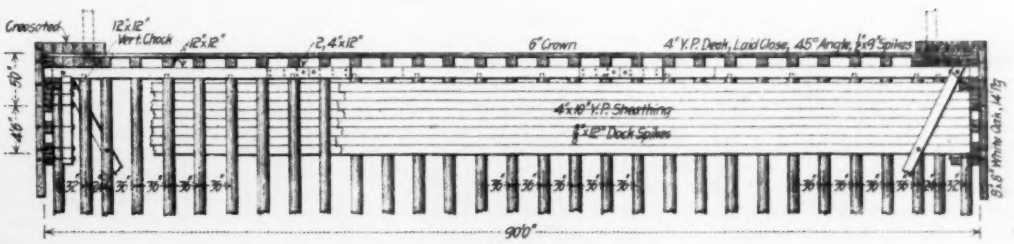


FIG. 12.
SHEATHED TRANSVERSE BRACING FOR PILES TOWARD END OF PIER, SPACED 3 FT. ON CENTERS.

FIG. 9.
ILS OF PILE
FORCED BY
NG TIMBERS.

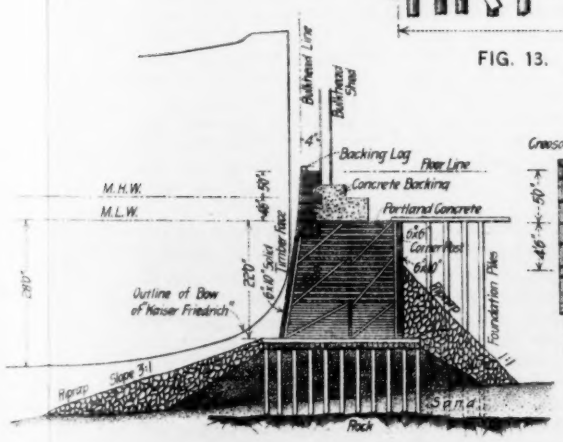


FIG. 2. TYPICAL TRANSVERSE SECTION OF BULKHEAD WALL AND FOUNDATION.

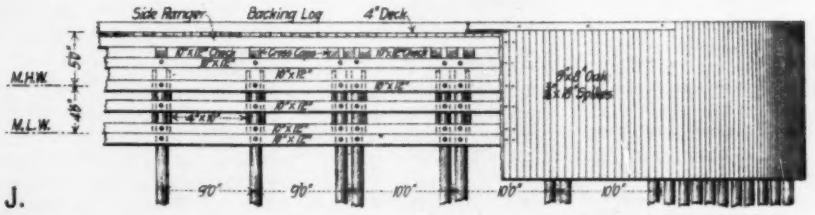


FIG. 18. ELEVATIONS SHOWING OUTSIDE SHEATHING FOR PIERS.

LLOYD STEAMSHIP CO.'S PIERS, HOBOKEN, N. J.

e, Assoc. M. Am. Soc. C. E., Engineer in Charge;
e, Jr., M. Am. Soc. C. E., Consulting Engineer;
H. Staats, New York, N. Y., Constructors.

CHAS. HART, LITH., 26 VESEY ST., N. Y.

The movement for uniform municipal accounts and statistics has extended within the last 18 months until now ten national and one state organizations have committees on the subject. The most hopeful sign of the movement is that the committees are empowered to co-operate for the common end in view. It is proposed that a committee from some strong society shall act as a leader, or central committee, and that all the committees work together to formulate schedules for uniform statistics in the various branches of municipal activity. Further information relating to this movement is given elsewhere in this issue, in our report of the recent meeting of the American Economic Association.

The two valuable papers in this issue upon tension impact tests should serve to direct renewed attention to this important field of engineering investigation. As has been frequently pointed out, what is desired in all engineering materials is not so much strength as toughness; or, to give toughness its scientific name, resilience. An impact or drop test determines this quality directly; while with the ordinary testing machine we obtain it indirectly by considering the strength in connection with the observed stretch of the specimen. Impact tests of finished parts, such as steel rails, car wheels, car couplers, etc., have long been common, but little has been done in the way of testing specimens of materials by impact in tension until very recently.

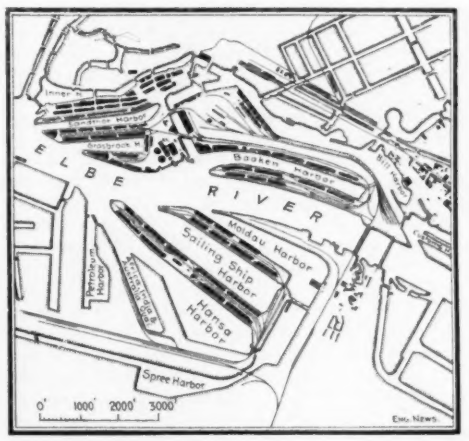
The ingenious machine of Mr. S. Bent Russell, M. Am. Soc. C. E., was described in our issue of Nov. 16, 1899, and in this issue Professor Hatt describes a much more complicated machine which has been constructed at Purdue University. For the benefit of busy readers it may be noted that in the Russell machine a swinging pendulum delivers the blow on the specimen, and the energy remaining in the weight is readily determined by measuring its upward swing after breaking the specimen. In the Hatt machine the weight falls vertically and its velocity before and after striking is measured.

The results which Mr. Russell has obtained in recent tests, and which he describes in this issue, are of special interest as the first public record, so far as we recall, of direct measurements of the comparative "ultimate resilience per cubic inch" of various materials. It is reassuring to find that in general the resilience by test was greater than the resilience by computation. In other words, it appears that with a fairly ductile material all parts of the specimen are more equally stretched under a direct blow than under a steady pull. Another fact of importance is that the high carbon steels showed much greater resilience than the samples of wrought iron, notwithstanding the high ductility of the latter. It may be noted that the resilience of the Norway iron was about the same as that found for mild steel in tests with the Hatt machine.

We pointed out two weeks ago, that one great difficulty in national subsidizing operations, such as the ship subsidy bill now before Congress, is that capitalists of other countries are likely to get the benefit and that there is no way of confining it to American capital alone. A timely illustration is furnished by the reported consolidation of Vickers Sons & Maxim, of England, the Cramps Ship & Engine Building Co. and the Midvale Steel Co. works, the whole to be known as the Cramps-Vickers-Maxim & Midvale Co., with a capitalization of $30,000,000, of which half or more will be held abroad.

FIREPROOF PIER CONSTRUCTION FOR NEW YORK HARBOR.

The decision of the North German Lloyd Steamship Co. to rebuild its Hoboken, N. J. piers, which were destroyed by fire under circumstances of unusual horror on June 30, 1900, marks a departure in pier construction in New York Harbor which deserves more than passing comment. These new piers are probably the most thorough and comprehensive attempt which has yet been made on this side of the Atlantic ocean to approach the solidity of construction and safety from fire usual to the pier and pier building construction of the large European seaports. In view of this fact it is next to impossible that the American work should escape a rigid comparison with the practice abroad in carrying out similar engineering constructions. It may reasonably be anticipated also that at first sight the American work will appear to be the chief sufferer in a comparison of this nature. Whether after a careful study of the conditions, however, this seeming inferiority of the American work will or will not prove to be a real one is a question which is open to considerable doubt; at least many engineers who are familiar with the local conditions in New York Harbor think that it can be defended to this extent that no one has yet come forward with a system of construction which is admittedly better suited to these conditions. To the majority of European engineers doubtless American pier and dock construction seems to be of an exceedingly flimsy and makeshift character. That this opinion is in a great measure just so far as the materials used and methods of construction commonly adopted are concerned most American engineers will be among the first to admit, but what many do refuse to admit is that the type of construction adopted is faulty when such conditions prevail as exist at the port of New York. It will repay us to consider briefly the structural features of the new



Sketch Plan of Portion of Hamburg Harbor, Showing Typical European Basin and Quay Construction.

North German Lloyd piers in the light of these seemingly very diverse opinions.

At the very outset of such a consideration it is necessary to set our minds straight upon the fact that the general plan of port construction and installation at New York and at American seaports generally is radically different from that of most large European ports. On this side of the Atlantic, and particularly in New York Harbor, the practice has been, as is well known, to plan the water front so that it consists of a series of piers seldom over 600 ft. long extending out from the shore at approximately right angles to the bulkhead line. These piers and the docks between them are given such dimensions as will enable them each to accommodate two vessels of the size usually arriving at the port. At New York the further condition exists that in Manhattan proper no railway line reaches the water front. On the New Jersey and Long Island shores a number of railway lines reach the water front, but there is no general extension of their tracks to other piers and docks than those owned by the railway companies themselves. As will be pointed out further on, this condition of affairs has an important influence on the means employed for handling cargo at New York as compared with the means employed abroad.

So much for the general plan of port installations at New York. In European ports generally the arrangement consists of a number of very large basins, the banks of which are formed of long and wide quays. These quays will vary from perhaps 200 ft. to 500 ft. in width and reach often a length of several thousand feet. In a goodly proportion, perhaps the majority, of European ports, also, the railway lines reach the water front and form a complete system of tracks along the quays and bulkhead fronts. Indeed, the length of the quays practically necessitates this complete railway system in order to handle the freight. The accompanying sketch plan of a portion of the port at Hamburg, Germany, is an excellent illustration of the typical European basin and quay port, with the tributary railway track system. For a comparative study of American water front arrangement the reader has simply to examine any good atlas map of New York city to observe the striking difference. The relative advantages of these two types of port arrangement need not be entered into here; the only point which it is necessary to remember is that in building their new terminals at Hoboken the North German Lloyd Steamship Co. had of necessity, if for no other reason, to conform to the general port plan of the harbor.

We are now in position to compare the structural details of these terminals as they are described elsewhere in this issue, with the methods of working out the corresponding details abroad, and also to examine intelligently the reasons which were considered by the engineers of the American work to warrant them in choosing the design and construction which have been settled upon. For these purposes we may conveniently divide the work into: (1) the pier and bulkhead construction proper; (2) the pier and bulkhead building construction, and (3) the means and appliances provided for handling the cargo. In taking up each of these divisions for consideration we shall content ourselves with a mere statement of the reasons which have influenced the choice of design and detail for the salient features of the work without much attempt to support or criticise these reasons.

Perhaps the first feature of the new North German Lloyd docks which attracts notice is the fact that in spite of the evidently thorough attempt to secure fireproof construction it was deemed advisable to retain the pile and timber platform pier construction which received such general condemnation when the original piers were destroyed by fire last June. It deserves to be pointed out, however, that far more elaborate precautions against the damage of fire are taken in the construction of these new piers than are common in such structures. As will be observed from the drawings of the piers given elsewhere, they are sheathed at the sides and ends from the floor level to a point well below low water, so as to prevent the entrance of flames beneath the platform. To provide a similar protection to the top of the platform it is covered with a layer of concrete. For a timber pier these precautions are perhaps all that practical considerations will warrant. It is needless to say, however, that European quay construction, which consists of a solid earth filling between masonry quay walls, ranks on an entirely different standard as regards safety from fire.

The real question obviously is not whether the construction adopted for the North German Lloyd piers is about as perfect as can be devised for timber piers, but whether timber construction is warranted at all, not simply because of its danger from fire, but because of its temporary character. Were good foundations for masonry walls available at reasonable depths there would be no difficulty in settling this question at once, but foundation of this character are not available at most places, if at any place, on the New York water front, at distances of six and eight hundred feet from the shore. Those who are familiar with the harbor bottom conditions of the North and East rivers point out that a very deep layer of soft mud forms the bottoms of those waters and that there are probably very few instances where the piles carrying the piers which line the water front do not depend entirely upon the skin friction to support their loads. Thus, although masonry piers may appeal to the engineer as a perfect construction for durability and safety from fire, it is agreed that they are of very doubtful practicability for New York Harbor.

Turning now to the only other possible substitute for timber, namely, metal structures, the en-

gineer at once encounters the question whether any practicable metal structure will present enough greater durability to warrant its extra cost over timber. In such great depths of mud as underlie the North River, steel screw piles are about the only type of construction which commends itself as at all suitable to the conditions. Supposing a screw pile pier carrying a steel and masonry platform to be successful in all other respects, the question still remains whether corrosion by the sea water will not cause it to require repairs and renewal as soon as a timber structure. Opinions on this question will undoubtedly differ, but the engineers of the North German Lloyd piers concluded, after careful consideration, that the advantages in favor of a timber structure were such as to warrant its selection instead of one of metal.

The provisions made for sheltering and temporarily storing cargo in America consists always of a pier shed covering practically the entire area of the pier, and when there are several piers belonging to the same terminal these sheds usually terminate in a bulkhead building on shore. This is in all essentials the construction adopted for the new North German Lloyd piers. In European ports the corresponding buildings usually comprise open transit sheds running along each side of the quay, with a space between the edge of the quay and the sheds on each side of from 30 ft. to 60 ft. and a wider space along the center of the quay between the two rows of sheds. In some instances a warehouse building is built instead of a transit shed, but more usually such buildings are located back of the lines of sheds near the center of the quay. The transit shed of European ports is usually a light iron or wood structure with a roof carried by columns and open at the sides. The warehouses are, however, steel and masonry fireproof structures of very substantial construction, with frequent fire partitions, inclosed fireproof stairways and elevator shafts, etc.

The pier sheds and bulkhead building of the North German Lloyd terminals at Hoboken are not exactly comparable with either the transit shed or the warehouse building of the European port; they partake somewhat of the character of both and approach perhaps the transit shed most closely in their purpose and the warehouse most closely in the solidity and fireproof character of their construction. Like the European warehouse, they are divided into compartments by vertical fire walls. In the bulkhead building these fire walls are of masonry, as they are universally constructed in Europe, but in the pier sheds they consist of tight boarded wood partitions sheathed on both sides with tin plate. Comparatively this is a much less efficient fire resisting partition than a good brick wall, but for lightness, availability, economy and efficiency combined it deservedly ranks high with American engineers, and it particularly recommends itself for a building to be erected on a pile platform pier. To place a masonry and steel fireproof structure of permanent construction on a substructure for which a maximum life of more than 35 years is not to be expected may reasonably be contended to be illogical.

It is, however, in their almost entire lack of equipment with elaborate and powerful mechanical appliances for handling cargo that the new North German Lloyd piers differ most strikingly from the quays of European ports. For a number of years the ports at Hamburg, Bremen, Havre and elsewhere in Europe have been working energetically to develop and improve elaborate systems of traveling and stationary cranes for handling ship's cargo both on the quay and from the vessel itself. This practice has been preached as the gospel of good engineering so widely that it has come to be believed by many engineers that the simple methods of loading and unloading vessels which prevail in American seaports were antiquated and inefficient. Naturally, therefore, when the intention of the North German Lloyd Co. to use none of the elaborate machinery of its home terminals in rebuilding its terminals at New York was announced, curiosity was aroused as to the reasons therefor. Shortly stated, they are simply that the company and its engineers consider that they can handle the package freight, constituting the bulk of the cargoes carried by their ships, more easily and efficiently and in less time by the simple gang plank and plain block and tackle method than they can by any installation of power cranes which has been brought to their attention.

It will be interesting to review briefly the reasoning of the North German Lloyd engineers in arriving at this decision. In European ports the transit sheds are set back from the edge of the quay sufficiently to permit always a single and often a double line of railway track. One of the chief purposes of the cranes installed along the edge of the quay is to handle the load from the ship's rail across these tracks to the transit sheds. This manoeuver is not required at the new Hoboken piers, since the ship's side lies so close to the side of the pier shed that a gangplank may be thrown across and a simple block and tackle used in combination with the ship's hoists for all the power hoisting operations necessary. The chief objection urged against a crane system, however, is that it is not and cannot be adapted to the handling of package freight. These packages vary in weight from a few hundred pounds to a dozen tons, and the crane will handle the light load no more rapidly or certainly than it will the heavy load. It is quite out of the question to try to meet this variability of load by varying capacities of cranes, since if anything is to be gained by this plan the cargo will have to be classified in its loading according to weights and dimensions, and this is not practicable. As a vessel ordinarily discharges its cargo a light load is quitely likely followed by a heavy load, a crate of French millinery will be succeeded by a sling load of Krupp locomotive tires. The method which best meets such variable conditions is one which can be immediately varied to suit the load, and no better method in this respect has been found than a large force of intelligently directed sailors and stevedores, aided only by such machinery as the ship carries and the gangplanks and blocks and tackles of the pier sheds provide. The prime object in loading and unloading vessels plying on busy ocean routes is to do it quickly and get the ship clear for its voyage; the loss of a single paying voyage a year by delays in loading and unloading amounts to more than is gained by even pretty large economies in handling freight which result in time being lost. It is claimed by the engineers of the new North German Lloyd docks that for handling package freight quickly and conveniently the elaborate crane equipment of European ports has no place on this side of the ocean.

One other point remains to be noticed particularly. The freight going from and coming to the new piers at Hoboken has all to be trucked to and from the business houses and railway stations. This explains the method adopted of driving the trucks directly into the pier sheds to receive and deposit their loads. In conclusion we wish to direct the attention of engineers to the fact that the builders of these first carefully planned fireproof piers in New York Harbor set themselves on record as directly in opposition to the notion that the large mechanical power plant of the European quay shed has a field on the short, narrow piers of American ports. It may reasonably be expected, we think, that there will be engineers who will dispute the wisdom of this position and perhaps be able to refute its accuracy, but, however this may be, we think that all will find food for reflection in the decidedly contradictory attitude assumed by a great German steamship company toward the power crane as a seaport appurtenance for which its home country is notable above, perhaps, all others.

LETTERS TO THE EDITOR.

Economy of the Walker Smokeless Furnace.

Sir: In your issue of Dec. 20 the data given of the performance of the Walker boiler setting do not include some recent tests made with return tubular boilers. These boilers were 18 ft. long by 6 ft. in diameter, and were rated at 124 HP. The following results give evaporation per lb. of combustible from and at 212° F.:

		Boiler driven over rated capacity.
Slack 11.16	lbs.	34.9 %
Soft coal, lump..... .....12.37	"	71.12%
Pea coal, hard ..... .12	"	58.3 %
Egg coal, hard ....over 12	"	57.2 %

There were no data given in your article of the results obtained with the puddling or heating furnaces. I beg to submit the following figures:

The average puddling furnace delivers 0.87 lbs. of iron per lb. of Pittsburg coal used; whereas with the Walker furnace the average is 1½ lbs. of iron per lb. of coal. The two heating furnaces referred to at the Keystone Bridge Works, previous to my changing them, turned out in 24 hours 44 eye-bars; this was the largest number they ever did. After the change as many as 119 were done in the same length of time with 50% less coal.

With the Walker continuous-heating furnace at another plant the average output per week was 1,032,640 lbs., and the coal per ton of billets heated was 152 lbs. The furnace before changing burned 400 to 440 lbs. of coal per ton of steel billets, 4½ × 4½ × 54 ins., and turned out per week 674,244 lbs., according to the statement of the owner of the mill.

Heating iron piles, the Walker furnace in one week's trial turned out 289,756 lbs., or 2.76 lbs. per lb. of coal. The same furnace before the change turned out 1.73 lbs. of iron per lb. of coal, thus showing 61% saving in favor of the Walker furnace. At the same time, 90% of the smake was got rid of. Robert L. Walker, Patentee.

Park Row Bldg., New York city, Dec. 22, 1900.

Subsidies for American Merchant Vessels.

Sir: In reading your editorial on this subject in Engineering News of Dec. 20, I note your quotation from Senator Frye's speech as follows:

Is the committee correct in its statement that it costs more to build and more to operate an American ship than it costs our commercial rivals to build and operate theirs? If the committee is mistaken in this respect, then the foundation of this bill falls through.

Your answer to this is, that it undoubtedly cost more in the past to build ships in the United States and to operate them under its flag than it did other nations.

I have within the past couple of years secured prices on the cost of building vessels from nearly all of the larger shipbuilding yards in Europe and America. In each case bids were made on one general standard specification without going into details, leaving these for each yard to put in for itself. The cost varied from $250,000 in Norway to $310,000 in England, and the lowest bid from an American yard was $425,000. The vessel specified was an ocean "tramp" capable of averaging nine knots. European builders were willing to guarantee this speed on a consumption not exceeding 25 tons of coal a day. The American builders calculated on 40 tons a day for the same service and results.

The point in this, and one which has not been touched upon, so far as I have seen, in any of the arguments advanced for or against the Ship Subsidy Bill, is that in European countries nearly all of the shipbuilding yards have standard drawings and standard specifications for "tramps" of various sizes. They have in their yards a set of patterns, and upon receipt of an order these are used. In other words, the ship possesses no individuality, except such as may come from the very slight modifications in the construction. They are all about alike. In America we have not had a demand for "tramp" vessels such as they have in Europe, and there is not a single shipbuilding yard which has such patterns. Owing to this, every vessel, whether a 'tramp" of the most ordinary type or a high speed boat, requires a set of patterns and detailed drawings from the beginning to the finish. The result will undoubtedly be better for the American vessel. It will possess individuality. It will be made according to certain specifications; and in this same matter of specifications, the wording of our contracts is far more clear and specific than the contracts of any other country. It is not necessary here in specifying the quality of steel to be used to say "the best, best steel." If we say the "best steel" there is no fear that an inferior grade will be used.

For the reason above given a low speed cargo vessel cannot help but cost more here than abroad; but as you come to a comparison between vessels of 15, 18 and 20 knots, the difference in cost decreases, and I think it will be found that to-day the cost of building a fast steamer of 22 or 23 knots will be little, if anything, more here than at shipbuilding yards in England or Germany.

The unfortunate part about the Subsidy Bill is that it does not make sufficient allowance for this difference. The subsidy increases as the speed of the boat becomes greater, while, in fact, these are the boats which have the least reason for having such subsidy. On the contrary, the "tramp," which is purely a freight vessel and carries the bulk of the world's commerce, receives an exceedingly small benefit and one which will not be enough, as the bill stands at present, to justify its operation under the American flag. Yours very truly,

"Bulk Freight."

New York, Dec. 26, 1900.

(Our correspondent might hit the "speed premium" feature of the subsidy bill a good deal

order and still be within the mark. As the bill stands, the subsidy is set at 1½ cts. per gross ton per 100 miles sailed for vessels of any class, and in addition a "speed premium" of ½ ct. for vessels of 12 to 14 knots, 1 ct. for 14 to 15 knots, 1.1 cts. for 15 to 16 knots, 1.2 cts. for 16 to 17 knots, 1.4 cts. for 17 to 18 knots, 1.6 cts. for 18 to 19 knots, 1.8 cts. for 19 to 20 knots, 2 cts. for 20 to 21 knots and 2.3 cts. for 21 knots or over. There is no good reason for these increases in subsidy with increase in speed, except where ships are used for carrying the mails; and this is such a minor and unimportant part of ocean traffic that to give increased subsidy for increased speed to vessels of any and all classes is utterly absurd. A vessel of 5,000 tons, sailing from New York to Liverpool at 11 knots would be paid under the subsidy bill $2,000 for the voyage. A vessel of the same tonnage making the same voyage at a speed of 14 to 15 knots would receive $3,770, or nearly double the amount. No public benefit is derived which justifies any such increase in the subsidy to the faster vessel.

As for our correspondent's remarks as to the relative cost of tramp steamers here and abroad, they merely go to show that in order to compete with European yards we must adopt the methods by which we have gained such success in other lines, and "manufacture" cargo vessels instead of "building" them. It is because we have "manufactured" machine tools and cranes and locomotives and guns where the foreigner has "built" them, that we have been enabled to undersell him in the world's markets. Surely we can apply the same methods to shipbuilding.—Ed.)

A Libel on American Bridge Work.

Sir: It was with extreme disgust that I read in to-day's despatches from London of the infamous and childish assertions concerning the honesty and skill of American bridge builders and engineers, credited to Sir Alfred Hickman.

I quote from the "Herald" of Dec. 20, as follows:

London, Dec. 19.—An agitation to bar American manufacturers from securing bridge and railway contracts from the British Government has been begun by Sir Alfred Hickman, M. P., a prominent steel maker of Wolverhampton and former president of the British Iron Trade Association. Sir Alfred maintains that much of the steel and iron exported from the United States is of inferior quality, and that American made bridges are of faulty construction. He has furnished the Foreign Office with statistics showing that on an average twenty bridges in the United States break down every year, and urges the necessity of a rigid inspection of the great viaducts and trestles which American engineers have recently put up in Burmah, Uganda and the Soudan.

In an interview to-day Sir Alfred said:

"There is one striking contrast between the British and American bridge builders. We expect bridges to last, regardless of the cost. The Americans proceed on the theory that they have done their work when they have built bridges to answer the purpose for the least possible amount of money. We do not believe in economy at the expense of stability."

Although Sir Alfred's purpose is so obvious that his statements do not merit notice, I am impelled to ask the use of your columns for a reply, especially as these attacks on Americans, in all classes of engineering work, are becoming quite common from disgruntled British manufacturers.

Any fair-minded person, who has studied the works of American bridge builders, cannot fail to admit that every element, every rivet, every brace, is a silent but convincing refutation of Sir Alfred Hickman's statements; and there is not a bridge from any reputable American builder that will not compare favorably with any English bridge for the same duty. Moreover, if it is of recent construction, it will show smaller first cost, simpler design, and greater economy of material than the English bridge of the same strength and stiffness.

As to the skill of such engineers as Baker, Fox, Pearson and men of that class I raise no question; but after an opportunity to observe the methods of the ordinary, every day English engineer, I have no hesitancy in stating that the rank and file are sadly deficient in scientific training, as compared with Americans.

As an instance of English methods, a case comes to my mind where a well-known London structural steel firm undertook to construct a certain type of belt conveyor after plans furnished by the American manufacturer. The American engineer who designed the apparatus requested the English firm to furnish him with the sizes of steel shapes they carried in stock, capacity of their machine tools, sizes of rivet dies and various other information concerning their facilities, having in mind the desirability of designing the work to facilitate the contractor's work and at the same time fulfil its ultimate purpose. When the plans were delivered to the contractor, his stock clerk found that he did not have the steel after all that he thought he did, his machine shop foreman discovered that his lathe face-plates wobbled; and the American wondered how the contractor made money and his executive force held their jobs. The contractor calmly informed our American friend that it would take six weeks to secure the steel shown in the plans, so the only alternative was to use what the contractor had on hand. These shapes, however, had 25% greater section, and this added 25% dead weight to the trusses in which it was to be incorporated while there was no room to increase the diameter or number of rivets. The contractor's engineer calmly asserted that the trusses would be stronger because they had more metal in them!

This particular work was designed with 10,000 lbs. tension and compression in chords, ties and struts at their weakest cross-section and all members simply cut out of stock bars; yet delegations of Englishmen would gravely comment on the "cob-webiness" of the steel work. Perhaps, however, English spiders do spin webs of 2½ × 2½ × ¼-in. angle iron.

As an example of the honesty of the British manufacturer, in these same structural works it was customary if two bolt holes did not come fair, in assembling a roof truss or floor beam, to just slip in any size bolt that would go in. Bolt holes were drilled with 1-16-in. clearance; trusses were bolted unless riveting was insisted upon; such a thing as reaming a hole after punching was a wild idea of a "theoretical" man. The catalogue of this firm contains the legend: "Strength of our beams, girders and columns not guaranteed unless by special agreement."

Coming back to bridges again, I wish to say that every railroad bridge over the Thames at London is in a disgraceful condition, and none of them seem to be on speaking terms with a good coat of paint. An English engineer informed me that the Tay bridge blew down because it was 50% rust; and if its condition was equal to that of some of the L., C. & D. spans I do not doubt it.

The trouble with Sir Alfred Hickman & Co. is that they have not kept up to date and they now are mortified by seeing even their own countrymen buying in a market where they get better value for their money; not value in raw material, but value with which the finished product is endowed by American brains, American artisans and American honesty.

Charles Jay Seibert.

640 Putnam Ave., Brooklyn, N. Y., Dec. 20, 1900.

The Shrinkage of Earthwork.

Sir: There are so many young men who read your valuable journal that I do not feel that it would be fair to them to let the discussion on shrinkage stop where it now rests. Experience has taught the engineers who build railroad embankments that all earth embankments settle more or less, as is proven by the fact that nearly every road in the west provides for "shrinkage" in its specifications. Many have found out to their sorrow that this settlement does not take place at once; as in some instances, embankments have been put up 10% of their height above sub-grade, and much of this has been cut off again ahead of the tracklaying.

The whole question as to the amount of shrinkage which will take place depends on two things: the character of the materials which compose the embankments, and the method by which this material is placed. There are also some minor considerations which enter into the subject, such as rainfall, depth of borrow pits, depth to which frost commonly penetrates, and the rainfall conditions while the work is in progress and afterwards.

Materials shrink in embankments in the following order: Sand and sandy gravel least; clay and clayey soils intermediate, and loams most of all. Methods of handling give results in the following order: Banks built with slush scrapers shrink least; and those built by other methods about in the following order: (1) With wheel scrapers; (2) with wagons; (3) with cars, and (4) with wheelbarrows.

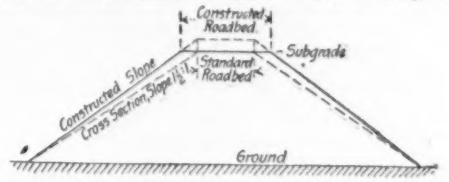
Rain has a very great effect upon settlement, and embankments put up in a very dry season will settle very much when soaked with rain; while embankments put up in very wet seasons settle but little after construction. The soil and other material taken from near the surface of the ground, in countries where frost enters the ground, are in a more porous state than materials below the reach of the frost, and cause greater settlements in the embankments.

From the above it is readily seen that the possible amount of shrinkage in embankments in general cannot be embodied in any rule. Neither do I believe that the height of any embankment should be increased during construction. For if the embankment does not settle, it not only leaves an "unsighty hump," but may even limit the trains where it occurs on a maximum grade, some distance above its foot. The settlement of embankments, in any case, is vertical, and there is no need of widening the base of the embankment to provide for shrinkage.

My method on the Union Pacific R. R., on the high arid plains in Wyoming, was to have the embankments cross-sectioned, as if no allowance were to be made for settlement; then, when the method of handling the material by the contractors and the peculiarities of the material itself were known, the amount of allowance to be made was determined. The material was such that its peculiarities made but little difference; and as the country was arid, the frost had but little effect and rains none.

The general rule was to make an allowance of 3% for slush scraper embankments, 5% for wheel scraper and wagon work, and 7% where cars were used and temporary trestles erected by the contractors to run the cars upon.

The method of making this allowance was novel (so far as I am aware it was never used before), and it was as follows: The slopes were steepened until the top width of the embankment at sub-grade was the same as if the embankment had been built to a height above sub-grade, equal to the percentage of its height which the method of construction warranted, and then cut down to sub-grade.



This is equivalent to the common practice of finishing the roadbed at standard width, at such height above sub-grade as is deemed best; and it avoids its disadvantages. This in no case increased the slope beyond 1.40 to 1, and we had no trouble in maintaining this slope, which would flatten as the embankment settled.

The question may be asked, what did we gain by this? Briefly, we had an embankment which when it settled would be wide enough to hold all the filling required to raise it to sub-grade, up to the limit of what our judgment told us would be the maximum. And as sags are no disadvantage within reasonable limits, we could haul the maximum tonnage as soon as the track was laid and ballasted; which more than compensated for the loss due to the contractor not putting in the earth required to bring the embankment to sub-grade after settlement.

Yours truly, L. B. Merriam.

De Kalb, Ill., Dec 17, 1900.

Sir: Relative to the shrinkage of earth embankments after construction, I desire to quote from a recent letter written to me by Mr. Chas. R. Felton, City Engineer of Brockton, Mass. He says:

In 1896, many earth embankments were built here for the abolition of grade crossings; the fill usually ran up to about 20 ft. at the bridge. They were filled with carts, and the material was usually sand or gravel. Edgestones were set on these embankments, which were not rolled, soon after their completion. Since this discussion I have had levels taken on two of these streets every 50 ft., and find a maximum settlement of 0.18 ft. I then had levels taken on the edgestones of a crossing built in 1892 under similar circumstances and find the following settlements:

Fill in ft.	Settlement in ft.	Settlement nearest quarter, %
5	.07	1¼
6	.05	1
7	.14	2
8	.08	1
10	.22	2¼
12	.05	¼
13	.17 to .23	1¼ to 1¾
15	.17	1
16	.30 to .36	2 to 2¼
18	.27	1½
19	.32	1¾
19	.46	2½
20	.47	2¼

Great care was taken to select points whose exact original elevation was known.

I think that your assumption of 2% settlement is warranted and will represent a probable maximum settlement, although under circumstances where the filling is simply dumped loosely without compacting effect of horses' feet, etc., as was the case where the edgestones settled, I think 3% might be possible. Of course, if you are filling on mud there is no practical limit to the settlement, and it seems to me that some of those deductions must have been drawn from filling on such material.

In a later letter Mr. Felton adds:

I think we may say, at least until further evidence is put forward, that banks for streets built of sand and gravel by means of horses and carts, will not settle more than 2½% of the vertical fill after completion, while at points inaccesible to the teams where dirt must be thrown by shovels or dumped without compacting, a 3% settlement may occur; although the average settlement where some care is taken to fill in layers should be much less than this.

The data that Mr. Felton has furnished are so complete and cover such a range of depth of fill, besides being based upon fills that have existed 8 years, that whoever may have been inclined to doubt the accuracy of my own observations as originally given, must concede that there was good reason for criticising a general rule allowing 16⅔% for shrinkage.

Prof. Johnson fell into a very natural error in formulating his rule, and with the openness of mind that characterises followers of a scientific profession only—and them not always—he has yielded to his critics in so gracious a manner that they can't but feel a tinge of regret at having written a criticism at all.

Yours truly. H. P. Gillette.

Rochester, N. Y., Dec. 29, 1900.

Notes and Queries.

An error, due to a mistake in the information furnished us, was made in describing the timber ore dock of the Great Northern Ry., in our issue of Dec. 20. There are 250 ore pockets in this dock, each having a capacity of 250 tons, making a total capacity for the dock of 62,500 tons.

In the actual working of the dock, however, it has been found that 70,000 tons of ore can be stored at once.

C. G., Brooklyn, N. Y., desires a practical formula for calculating the load that an iron ring of mean diameter D and of circular cross section of diameter d can carry without permanent deformation, or without the fiber stress in any part of the ring exceeding the elastic limit.

Our attention is called to an error in the discussion of Mr. W. H. Henby's tests on the elasticity of concrete in our issue of Dec 13, 1900. In referring to the difference between the tensile and compressive moduli of elasticity, the difference shown by the diagram was multiplied by ten in the accompanying text.

A correspondent requests information as to the best manner of paving railway stock pens with moderate first cost. The case in point is a stock pen of about two acres area, with very slight drainage grade (1% in one direction only). Particulars as to thickness of paving and a rude idea of cost are desired. Would furnace slag, rather porous, covered with 2 or 3 ins. of clayey loam, be good? Would fine gravel be better for this top dressing, or would gravel be better for the entire thickness?

We shall be glad to hear from our readers concerning these questions.

THE LARGEST GASHOLDER IN THE WORLD is being built by the city of Birmingham, England. The diameter of the tank is 300 ft., and its depth is 31 ft. 6 ins.; but it comprises six telescopic lifts, each of this height, and the full capacity of the holder is nearly 12,000,000 cu. ft. The engineer is Mr. Henry Hack, M. Inst. C. E.

THE BERNIER ARCTIC EXPEDITION has been organized by Capt. J. E. Bernier, a Canadian sea captain of many years' experience in all parts of the world. He hopes to enlist the British government in his attempt to reach the Pole by somewhat the same route taken by the ill-fated "Jeanette." With the belief that the Polar basin is a frozen ocean, Capt. Bernier proposes to leave the Siberian coast somewhers near the Lena River, and to make his way to the solid ice, which he thinks extends to the Pole.

THE UNBALANCED MAGNETIC PULL upon the armature of an electric dynamo or motor when the same is not concentric with the field, formed the subject of a paper recently read before the American Institute of Electrical Engineers by Mr. B. A. Behrend. He has worked the matter out mathematically and finds that the results thus obtained agree quite closely with those observed in actual experiments. They may be stated in a simple manner as follows: Substitute for the round armature a square one of equal perimeter, mounted in a square frame and with the same air-gap as the round armature. Consider this square armature to be displaced in a direction normal to one of its sides through a distance equal to the displacement of the round armature. Then the unbalanced pull upon the displaced square armature will be equal to the radial pull upon the actual round armature. The pull upon the sides parallel to the displacement will, of course, still be balanced. The unbalanced pull may easily be calculated, since it occurs between flat surfaces each equal to one-quarter of the surface of the round armature, by using the following formula:

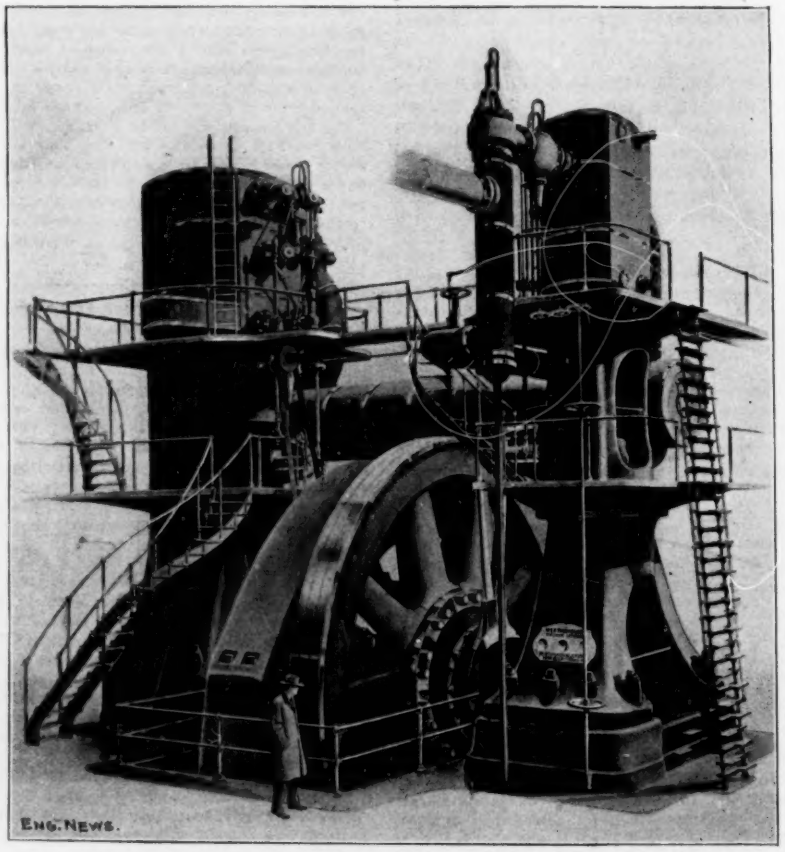
$$Z = \frac{S\,L^2}{8\,\pi} \cdot \frac{4\,\triangle}{\delta},$$

In which Z = the total pull in dynes, S = the area in square centimeters of the square surface, L = density in lines per sq. cm., $\triangle$ = displacement of the armature, and δ = the normal air-gap, both in centimeters.

4,000-HP. CROSS COMPOUND ENGINE FOR A BROOKLYN ELECTRIC LIGHTING STATION.

The large engine illustrated in the accompanying cut is one of two which are nearly completed by the Westinghouse Machine Co. of Pittsburg, Pa., for the Bay Ridge station of the Kings County Electric Light & Power Co., of Brooklyn, N. Y. The engine and generator have a nominal capacity of 4,000 HP., but are designed to carry for short periods overloads up to 7,500 HP. Although these machines are very large the builders have others under way which are even more powerful, including 16 engines of 5,000 HP., each for the Kingsbridge power-house of the Third Ave. Ry. of New York city, two more of the same capacity for the Boston Elevated Ry. and eight engines of 6,500 HP. each for the new power plant of the New York Gas, Electric Light & Power Co. The design of this engine has been prepared for the Westinghouse Machine Co. by Mr. Cyrus Robinson, M. Am. Soc. M. E., chief of the company's Corliss engine department.

The engine is of the vertical cross-compound condensing type, with intermediate reheating receiver and has been designed to drive an alternating current generator mounted directly upon the shaft of the engine. The alternator will deliver three-phase current at 6,600 volts and 3,000 alternations. The principal dimensions of the engine are as follows: Height from floor line to top of cylinders, 35 ft.; length along shaft, 38 ft. 6 ins.; width, 28 ft.; diameter of high pressure cylinder, 46 ins.; diameter of low pressure cylinder, 86 ins.; stroke, 60 ins.; revolutions per minute, 75; steam pressure, 170 lbs. per sq. in. The diameter of the shaft in the center is 39 ins.; diameter at bearings, 34 ins.; length of each bearing, 60 ins.; total weight of the shaft without crank cheeks or fly-wheel centers, about 75,000 lbs. The shaft is of hydraulically forged steel, bored out hollow to 16 ins. internal diameter through its entire length. The fly-wheel is very massive, being 28 ft. in diameter and having 100 tons in the rim. The rim velocity is 6,600 ft. per minute. The bearing



4,000-H.P. COMPOUND ENGINE, WITH DIRECT-CONNECTED ALTERNATOR.
Built for the Kings County Electric Light & Power Co., Brooklyn, N. Y., by the Westinghouse Machine Co., of Pittsburg, Pa.

shells and caps of the engine, and also the cross-head guides, are cooled by water circulation. The valves are of the poppet type and are controlled by an improved Corliss valve gear. Each engine is provided with two bed-plates, one of which alone weighs 43 tons. For transporting these castings the Pennsylvania R. R. Co. constructed special steel cars.

We are indebted to the builders of the engine for the information from which the above description has been prepared.

A COURT DECISION AS TO RESPONSIBILITY FOR DAMAGE BY ELECTROLYSIS TO GAS MAINS.

An interesting and possibly an epoch-marking decision relating to the responsibility for damage by electrolysis to underground pipes has just been rendered at Indianapolis, Ind. The decision is a preliminary one, in the Superior Court, overruling a demurrer in which the defendant sought to set aside the case, but inasmuch as the case is to be heard by the same Judge on its trial, and because the text of his decision covers some of the points at issue in the case itself, it is likely that the present decision lays down principles which will be followed later on.

The action in question was brought by the Manufacturer's Natural Gas Co., against the Indianapolis Street Railway Co., for damages done by its currents to the pipes of the gas company. Mr. F. A. W. Davis is manager of the gas company and we are indebted to him for the following abstract of the decision:

The method in use by the defendant in operating its cars results in serious injury and in some cases to the destruction of plaintiff's pipes. The defendant can, by the use of an approved appliance at reasonable expense so operate its cars so as to avoid injuring the plaintiff's pipes. The plaintiff cannot, by any known method, protect its pipes from injury.

The plaintiff owns its pipe line laid in the street by legal authority. Such pipe line is property within the meaning of the constitution and laws. It is subject to taxation; it may be sold and purchased as other property. The Street Railroad Company seizes on these pipes and makes use of them as a conductor for its return current, and in so doing greatly injures and in some instances wholly destroys them, and this is done under a claim that it is performing a public service under authority of law. Is not this a taking of private property for public use and for which just compensation must be made?

I do not see that the fact that the Street Railway Company's tracks, poles and wires may have been placed in the street before the plaintiff's pipes were so placed can make any difference unless it should be shown that when the plaintiff placed them there it knew of the injuries to which they would be subjected.

The city could not and did not grant a monopoly of the street either to the defendant or its predecessor, and when the tracks, poles and wires were placed in the street the Railway Company knew that gas and water pipes might be laid in the street at any time, and it acquired its rights to run an electric road subject to that fact, and all the consequences that might follow. The plaintiff is not a trespasser, but occupies the street lawfully, and while there its property is taken by the Street Railway Company as a consequence of its operations.

Justice Brown, in the case in 42 Fed. Rep. 279, states that the substances of all the cases which he has examined is that:

Where a person is making lawful use of his own property, or of a public franchise in such a manner as to occasion injury to another, the question of his liability will depend on the fact whether he has made use

the means which, in the progress of science and improvement, have been shown to be the best.

A street railway company is not, however, bound to adopt the latest invention, nor to adopt any before its utility and practicability have been demonstrated by use. The case made by the complaint in this cause seems to meet the requirement contained in Judge Brown's statement:

According to the allegations of the complaint, the defendant has failed to adopt an approved appliance which could be done at a reasonable expense, and when by so doing the injuries complained of could be avoided.

I cannot see any difference in principle between a case in which liability attaches from a failure to adopt the best known device and one where there is a failure to adopt any device whatever, when there are devices by which the injury could be avoided, and the utility of which has passed the point of being a mere experiment.

These questions are new, and we are without precedent in decided cases for our guidance, but as these new questions arise the administration of the law should keep step to the new situations arising in the march of scientific invention and improvement, not by inventing new legal principles, but by the expansion of old and well-recognized principles of law and equity so as to meet and cover the new situation. It would be a reproach to our system of jurisprudence and the administration thereof, if a situation could arise in which large and material injury should be done to legal rights and destruction caused to property and the law be powerless to apply a remedy.

Take the situation in this city. The number and size of the cars propelled through our streets by electricity is ever rapidly on the increase. Suburban lines are coming into the city from every direction with cars larger than ever before used on the streets, and the currents of electricity which now are, or soon will be discharged into the earth of the streets are very large, and if it be true, as claimed in some of the cases which I have examined, that these currents not only attack gas and water pipes, but the steel frames of tall buildings as well, and that such steel frames may be deteriorated and weakened so as to imperil such buildings, is there no remedy in the law to prevent it, or to compel the Street Railway Company to control its return current when it is conceded that it is reasonably in its power to do so?

Where a corporation is exercising a public franchise, and does so in such a manner as to cause actual material injury to legal rights and destruction of property when at reasonable expense, by the adoption of well-known and approved appliances, the injury could be avoided, and the person injured is powerless to prevent or guard against such injury, then I think it must be held to be negligence in the use of its franchise on the part of such corporation not to adopt such approved appliances.

The destruction of the plaintiff's pipes so as to permit gas to escape and cause destruction to life and property cannot be said to be mere annoyances, inconveniences or incidental damages. It has been held in New York, in the case of Siebrecht vs. East River Gas Co. (47 N. Y., Supp. 262, 1899), that the Gas Company is liable in damages sustained by the action of gas which escaped from the company's pipes from leaks caused by electrolysis; to be subjected to such a liability cannot be considered a mere incidental damage.

For the foregoing reasons I think the complaint states a good cause of action, and the demurrer is therefore overruled.

THE NEW FIREPROOF PIERS OF THE NORTH GERMAN LLOYD STEAMSHIP CO., AT HOBOKEN, N. J.

(With two-page plate.)

We illustrate on our two-page plate this week the general plans and details of the new fireproof piers, which the North German Lloyd Steamship Co. will build at Hoboken, N. J., to replace the piers destroyed by fire on July 30, 1900. The new piers are important in being perhaps the most perfect example of fireproof pier construction in New York Harbor, and probably in the United States. The plans for this work have been prepared by Mr. W. F. Whittemore, Assoc. M. Am. Soc. C. E., in consultation with Mr. Geo. S. Green, M. Am. Soc. C. E., formerly Chief Engineer of the Dock Department of New York city, and with Mr. Joseph P. Gray, Vice-President of the Boston Manufacturers' Mutual Fire Insurance Co., the New York Board of Fire Underwriters', and Mr. Hugh Bonner, formerly Chief of the New York Fire Department. We are indebted to the company's engineer, Mr. W. F. Whittemore, for the information from which this description of the new work has been prepared. The contractors for the piers and buildings are R. P. and J. H. Staats, of New York city.

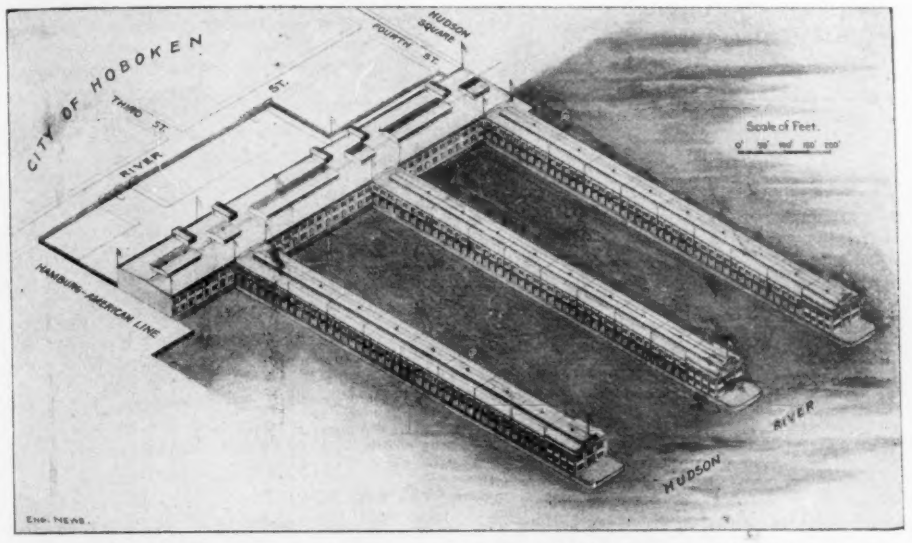


FIG. 1.—SKETCH SHOWING LOCATION AND APPEARANCE OF THE NEW NORTH GERMAN LLOYD STEAMSHIP CO.'S PIERS AT HOBOKEN, N. J.

W. F. Whittemore, Assoc. M. Am. Soc. C. E., Engineer-in-Charge; Geo. S. Greene, Jr., M. Am. Soc. C. E., Consulting Engineer; R. P. & J. H. Staats, New York, N. Y., Contractors.

Summarized briefly, the new work comprises the construction of a solid bulkhead 900 ft. long along the front of its property; the erection of a masonry and steel building on and behind this bulkhead; the extension of three piers out into the river at right angles to the bulkhead; the erection of steel sheds on these piers, and the provision of all the appurtenances to these structures. The sketch, Fig. 1, shows the relative locations and general structural characteristics of these different parts of the work.

BULKHEAD WALL.

The bulkhead wall, as stated above, will be about 900 ft. long, and it will extend along the water front of the company's property parallel and 329 ft. distant from River St., Fig. 1. A typical section of the wall is shown by Fig. 2. It is to be of masonry built with a timber caisson. To prepare a foundation bed for the caisson, piles spaced 3 ft. apart will be driven to hard bottom covering an area 30 ft. wide, the whole length of the wall. These piles will be filled between with fine cobble stone or coarse gravel to their tops which will be cut off at a height of 24 ft. below mean low water. The bottom of the caisson, which will consist of two courses of 12 × 12-in. timbers, will rest directly upon the top of the cobble-filled pile foundation. As will be seen from Fig. 2, this platform-like bottom of the caisson extends 3 ft. beyond the side walls of the caisson on each side. The walls of the caisson will consist of 6 × 10-in. yellow pine timbers laid solid with the greater dimension horizontal. To brace these walls together there will be a row of 6 × 6-in. timbers running lengthwise of the caisson, and also similar rows of timbers running across the caisson at 10-ft. intervals. For a height of 6 ft. above the inside bottom of the caisson the timbers of both the longitudinal and trasverse bracing will be laid solid to form virtually transverse and longitudinal bulkheads. Above this height these timbers will be spaced 6 ins. apart vertically. Fastened to each system of transverse braces there will be diagonal braces of 3 × 8-in. timbers running from the longitudinal braces to the side walls.

The interior of the caisson is to be filled solid with Portland cement concrete in which may be bedded clean, hard, angular stone, not larger than what is known as a two-man stone. The composition of the concrete used will be one part Portland cement, two parts sand and five parts broken stone. The concrete is to be carefully deposited and thoroughly rammed. As located, the top of the timber caisson will come just to the level of mean low water. Above this point the bulkhead wall will consist of an ashlar stone facing with a backing consisting of one part Portland cement, two parts sand and four parts broken stone concrete. The front of this wall is to be protected by a 12 × 12-in. yellow pine backing log fastened with expansion bolts to the stone coping, and with 6 × 10-in. vertical oak fenders, spaced 8 ft. apart. Finally the heel and toe of the wall are to be protected by riprapping.

As will be seen from Figs. 1 and 2, and more clearly by the transverse section, Fig. 4, the bulkhead building will have its water side close to the front edge of the bulkhead wall, and will be 850 ft. long and 130 ft. wide. The front wall of the shed will be carried by the bulkhead wall, but the other walls and the intermediate columns will have an independent pile foundation. The piles are to be not less than 10 ins. in diameter at point of cutoff, and 6 ins. in diameter at the point, and are to be shod with iron so that they may be driven through the riprap into the hard bottom. The sketch plan, Fig. 4, shows the method of arrangement of the piles in the foundation. On the piles there will be a platform or grillage consisting of 6 × 10-in. transverse timbers, carrying a longitudinal planking of 3-in. plank. On the platform over each transverse line of piles, or 20 ft. apart, there will be a brick cross wall, 4 ft. wide at the bottom and 2 ft. wide at the top, and capped with an 8-in. granite cap for supporting the ground floor I-beams, Fig. 4.

The frame of the bulkhead building will be of steel with built-up columns, plate girders and rolled I-beams, arranged to make spans of about 42-ft. 9 ins. across the building, and of about 20 ft. lengthwise of it. The plate girders run transversely across the building in the second floor and roof, with webs 3½ ft. deep and 6 × 6-in. angles

Fig. 3.—Sketch Plan of Piling for Bulkhead Shed Foundation.

and ⅜-in. cover plate chords. The I-beams will be carried by riveted brackets and will be 15 ins. deep. The ground floor I-beams, 24 ins. deep, will rest directly on the brick cross walls. Between the I-beams on both floors and at the roof there will be 4-in. brick arches covered with one natural cement, two sand and four broken stone concrete. This concrete filling will have a brick paving for the ground floor and roof and asphalt paving for the second floor. Details of this floor and roof construction are shown by Fig. 5. As shown by Fig. 4, the brick arch roofing is used for the two side bays only, the central bay being raised to form a monitor roof, and having a triangular roof

trussing carrying a roofing of 3-in. plank covered with five-ply felt and gravel covering. The exterior walls of the building are to be of brick laid in Portland cement mortar to a height of 4 ft. above the ground floor and in natural cement mortar for the remainder of the height. The interior columns of the building are to be protected from fire by a concrete casing constructed as shown by Fig. 6. Transverse fire walls, with double doors of wood covered with sheet iron, will cross the building at five points, and fireproof walls will also inclose the stairways and elevator shafts, as shown by the plan, Fig. 7. This plan

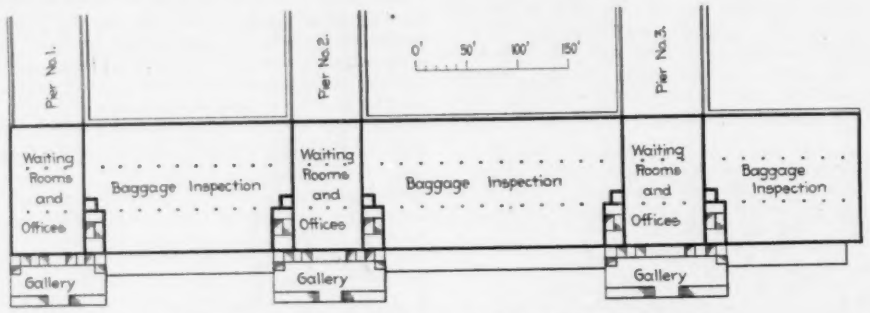


FIG. 7.—SECOND FLOOR PLAN OF BULKHEAD SHED.

shows the points where the pier sheds connect with the bulkhead building, and opposite these points on the shore side of the building the gallery extensions of the second floor, which are connected with the ground and also with the side bays of the roof by stairways. As shown by Fig. 4, there will be a hood or awning along the shore side of the building.

The ground floor of the bulkhead building will be used for storing the incoming and outgoing cargo, which will be removed and delivered by trucks. As will be seen by Fig. 1, a large yard is provided between Water St. and the building for the accommodation of carriages and trucks. The second floor of the building will be devoted wholly to the passenger traffic, the baggage rooms, waiting rooms and offices being all on this floor.

PIERS AND PIER SHEDS.

As shown by Fig. 1, three piers will extend out into the river from the bulkhead building, which will be known as piers No. 1, No. 2 and No. 3, pier No. 1 being the northernmost. These piers will be, respectively, 80 × 905.46 ft., 80 × 889¾ ft., and 90 × 869.27 ft. The piers are to be pile structures, with the piles spaced 6 ft. c. to c. in transverse rows, 9 ft. apart, except at the outer end, where for a distance of about 300 ft. the piles are to be spaced 3 ft. c. to c., and the five outer rows made double. The part sketch plan, Fig. 8, shows

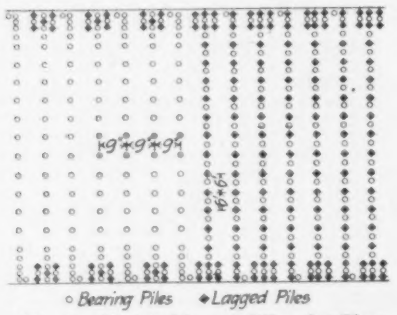


Fig. 8.—Sketch Plan of Piling for Piers.

the two arrangements. Ordinarily the piles used will be ordinary 85-ft. to 90-ft. piles, 14 ins. in diameter at the point of cutoff and 6 ins. in diameter at the points, but at certain points, as indicated in Fig 8, lagged piles will be employed. Details of one of these lagged piles are shown by Fig. 9. The outer ends of the piers are to be built with round corners, as shown by the detail drawings in Fig. 10.

A plan of the platform framing is shown by Fig. 11, and Figs. 12 and 13 show typical sections across the pier. The caps, stringers, backing logs, sills and subsills will be 12 × 12-in. timbers, and the deck planking will be 4 ins. thick. The deck planking, sills, subsills and the several courses of timber laid solid at the sides above and below the deck will be creosoted. The close rows of piles toward the end of the pier will be braced with 4-in. sheathing on each side from low water up to the side cap. This construction is shown by Fig. 12. The bracing of the remaining rows of piles consists of a double horizontal A-bracing, with the side piles further braced with diagonal sheathing above the A-braces, as shown by Fig. 13. At the extreme outer end of the pier, where the five outer rows of piles are driven double, the transverse bracing will be horizontal lagging, and wales, as shown by Fig. 14, and the rows of piles under the cross partitions or fire walls (A) of the pier shed, will be inclosed on both sides with vertical sheathing arranged as shown by Fig. 15. Fig. 16 shows details of the framing at points where derrick masts are located, and Fig. 17 shows details of the mooring posts and their foundations. Finally, Fig. 18 shows the method of inclosing the sides and outer end of the pier. The outside oak sheathing is grooved for 2 × 4-in. yellow pine splines. The details shown in Figs. 8 to 18, inclusive, are for pier No. 3, but the other two piers will be exactly similar in construction, except that the framing of pier No. 2 will provide for carrying a 3-ft. 4-in. sewer its entire length at the center and below the platform. This sewer extension will be built of 4-in. creosoted staves with galvanized iron clamps.

Each pier will be surmounted by a two-story steel frame pier shed occupying the entire width of the pier, with the exception of 4 ft. on each side. The sheds for piers No. 2 and No. 3 will have two floors, as shown by Fig. 19, but that for pier No. 1 will have at first but one floor, as shown by Fig. 20. For piers Nos. 2 and 3 the shed construction will consist of two wall columns carrying a plate roof girder; the roof girder supporting a monitor ventilator and having suspenders

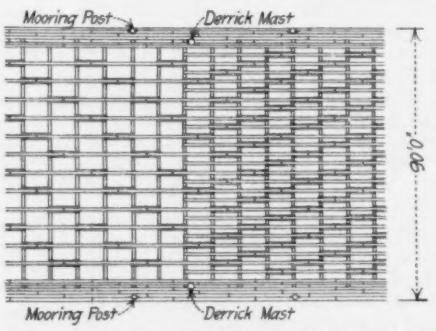


Fig. 11.—Sketch Plan Showing Platform Timbering for Piers.

which, with the wall columns, support the second floor girders. Fig. 21 shows a section of one of the wall columns with its fireproofing. The bents for all of the piers will be spaced 18 ft. apart. The side walls between columns will consist of wood sheathing covered with tin, and the roofs will consists of 3-in. tongued and grooved sheathing covered with felt and gravel roofing. The second floor will consist of 3-in. grooved flooring of yellow pine, covered with three-ply roofing felt and asphalt roofing cement to prevent the passage of water, on top of which comes a thickness of asbestos felt and then a wearing floor or surface of ⅞-in. × 2½-in., tongued and grooved maple flooring. The ground floor of the sheds will consist of a 6-in. layer of concrete, in which will be bedded 4 × 6-in. oak nailing strips for fastening a 2 × 8-in. oak flooring. As a protection against fire the undersides of the floor and roof of the pier sheds will have No. 22 gage galvanized corrugated iron arches sprung between the bottom flanges of the floor beams, and filled with cinder concrete, with enough cement to prevent the formation of dust and the disintegration of the filling.

As a further protection in case of fire each pier shed will be divided into three nearly equal portions by two transverse partitions or fire-walls. These walls will consist of three thicknesses of tongued and grooved boards of a maximum width of 4½ ins., covered on each side with small roofing terne plates, about 10 × 14 ins., locked and blind-nailed and painted on both sides before being applied. The openings through these cross walls for the passage of freight and passengers, two on each floor, and the openings in the side walls are to be closed with tin-covered wooden doors of the standard underwriters' type. All windows, including skylights, will have iron sash and be glazed with wire-glass.

FIRE APPLIANCES.

For protection in case of fire the underside of the roof and the underside of the floors above the first floor in both the bulkhead shed and the pier sheds will be fitted with automatic sprinklers, on the dry pipe system, of a construction approved by the Board of New York Fire Underwriters. In addition a line of 6-in. water pipe will be run along each side of each pier, with hose connections and 50-ft. reels of hose located every 50 ft. Finally, the entire bulkhead and pier structure will be provided with the multiphase cable system of automatic alarm in case of fire, with loose coils of the same cable to be laid over stored merchandise.

MISCELLANEOUS.

The sides of the piers will be provided at intervals with derrick masts 61 ft. in height, 15 in. in diameter at the heel, and 14 ins. in diameter at the top band, fitted with steel span wires and back stays for handling freight. At intervals of 90 ft. cast-iron mooring posts, of the construction shown by Fig. 18, will be provided. The various structures are to be provided with gutters, and suitable finishing and finally painted. All stairways are to be of fireproof construction, and provided with coverings.

TENSION IMPACT TESTS OF ROLLED STEEL.

By S. Bent Russell, M. Am. Soc. C. E.*

In Bulletin No. 5, Oct., 1899, of the American Section of the International Association for Testing Materials is given a Preliminary Report on the Present State of Knowledge Concerning Impact Tests. This report shows that several experimenters have been making tension tests of metal by impact in recent years. In only one case does it appear that these tests were each made by a single blow in such manner that the energy absorbed by the test piece could be measured. This case is that of experiments made at Purdue University with a hammer falling vertically. On Oct. 4, 1899, a paper was presented to the American Society of Civil Engineers† which described experiments made by the writer of this article. The experiments were tension impact tests of iron and steel where the test piece was nicked in such a manner that only the metal lying very close to the plane of rupture was materially distorted. The specimens were each broken by a single blow, and the amount of energy absorbed in rupturing the piece was measured. The blow was struck by a hammer swinging like a pendulum.‡

In this article the writer will give the results of some experiments made in the same way, but with a greater length of reduced section in the test piece. The specimens were cut from flat bars and the reduced section was always 1 in.

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†Impact Tests of Structural Steel, Trans. Am. Soc. C. E., Vol. XLIII. An abstract of this paper appeared in Engineering News of November 16, 1899.

‡A cut of this machine is given in Engineering News of November 16, 1899.

TABLE I.—Showing Results Obtained with Five Different Melts of Bessemer Steel Tested by Impact.
(Each resilience value is the mean of two tests.)

No.	Thickness of plate.	Tensile resilience, in.-lbs. pr cu. in.	Ultimate tensile strength in lbs. per sq. in.	Per cent. Elongation in 8 ins.	Per cent. Reduction of area.
F.	0.30-in.	26,100	64,600	26.4	52.0
G.	.25-in.	22,900	63,400	27.5	55.2
H.	.30-in.	25,800	63,600	26.2	51.8
I.	.30-in.	25,100	62,100	25.7	54.0
J.	.30-in.	27,400	60,600	26.2	53.5

TABLE II.—Giving Values of Tensile Resilience Obtained With Different Grades of Wrought Iron and Steel Tested by Impact.

Material.	Lot No.	Thickness of plate in. ins.	No. of tests.	Tensile resilience, in.-lbs. per cu. in.	No. of tests.	Tensile resilience in in.-lbs. per sq. in.*	Ultimate tensile strength, lbs. per sq. in.	Per cent. Elongation in 8 ins.	Per cent. Reduction of area.
Iron, Tennessee charcoal	4	0.25	4	17,900	6	2,921	52,500	27.5	52.0
Iron, Tennessee common	3	.25	4	10,500	6	2,320	55,000	21.2	22.5
Structural steel	19	.25	8	25,300	4	6,390	56,700	25.6	60.2
" "	25 A	.37	1	26,200+	1	6,500	61,330	27.5	57.5
" "	25 B	.31	1	28,200	1	8,600	62,400	27.5	57.0
" "	25 C	.31	1	28,200	1	8,640	60,900	28.7	59.4
" "	25 D	.18	1	27,200	1	5,390	65,000	25.0	48.0
" "	25 E	.29	1	28,000	1	5,860	63,100	32.0	48.0
Plow steel	26 A	.30	2	20,800	}				
Plow steel	26 A	.30	2	10,250	} 6	2,213	129,700	11.0	20.8
Structural steel, Bessemer	27 A	.19	1	24,400	1	4,650	64,500	25.6	50.5
" "	27 B	.31	1	27,500	1	7,890	62,600	28.7	60.0
" " Bessemer	27 C	.44	1	27,700	1	7,840	62,000	27.5	55.0
" " "	27 D	.35	1	27,100	1	8,140	62,100	27.0	55.8
" " open-hearth	27 E	.35	1	26,700	1	8,240	62,700	28.7	58.5
" " Bessemer	27 F	.26	1	27,000+	1	6,680	63,000	27.0	50.2
" " "	27 G	.24	1	27,400	1	6,660	63,000	26.2	59.2

*With sharp nick.

length by ¼-in. in width, while the thickness varied with the metal. Fig. 1 shows the form and dimensions used. The specimens were each carefully calipered and then placed in position in the testing machine and pulled in two by a single blow of the hammer. The energy absorbed in rupturing the piece was computed in inch-pounds from observations made on the testing machine. The swinging hammer used weighed 203 lbs., and the fork bar weighed 3.86 lbs. The area of the reduced cross-section multiplied by the length of 1 in. gave the volume of metal tested. The inch-pounds of energy absorbed divided by the volume of metal gave the ultimate tensile resilience in inch-pounds per cubic inch.

Table I. gives the results obtained with five different melts of Bessemer steel. The quality of the steel as shown by the ordinary tests is given in the last three columns. Table II. gives values of tensile resilience obtained with different grades of wrought iron and steel tested as above described. In another column for comparison we have values of resilience obtained where the specimen was cut with a sharp nick on each side, so as to give the minimum volume of metal in the

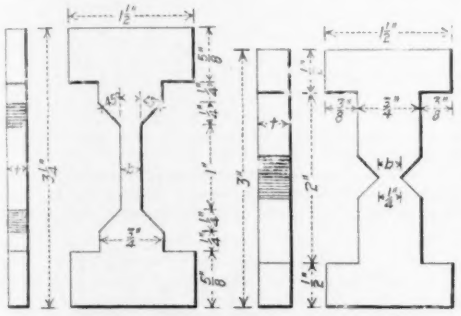

Fig. 1. Fig. 2.

Fig. 1.—Impact Tension Test Specimen With Long Nick.

Fig. 2.—Impact Tension Test Specimen With Short Nick.

field of distortion. The specimen was practically the same shape as in the former case, except that the length of the reduced section was shortened to zero. Fig. 2 shows the form and dimensions of these test pieces. It may be seen from the table that the resilience per cubic inch with the long neck ranges from 10,250 to 28,200; while the resilience per square inch with the sharp nick ranges from 2,213 to 8,640.

Fig. 3 is a photograph of a number of specimens. Three of these test pieces failed to break under the blow. That they were nearly ruptured is shown by the necking down of the metal. This necking down often occurs at two points. The specimen with square shoulders shows this to a marked degree. The specimen in the lower right-hand corner was made with sharp nicks. The whitish appearance of the specimens is due to their having been rubbed over with chalk for marking purposes. The distortion of the pieces sidewise, which will be noticed in the specimens at the top of the picture, is due to the dropping down of the fork bar and is not observed except where the specimen fails to break.

Table III. contains the most important matter to be presented here. The values given in this table were obtained in this way: From a bar of iron or steel six test pieces were cut of the same form and dimensions, the shape being that shown in Fig. 1. Four of these specimens were broken in the impact machine. The other two were tested by applying a steadily increasing load with a testing machine and measuring the elongation with an extensometer. The loads and deformations were plotted and from the area of the diagram the energy absorbed or ultimate resilience was computed. It should be remarked that the curve of stress and deformation could be determined very well up to the point of maximun. load, but beyond that point but few points on the curve could be gotten with the appliances used The latter part of the curve had to be assumed more or less, and, therefore, the area of the diagram only approximately represents the true resilience. It is thought, however, that the error from this cause is not large enough to leave the results without value.

The following brief description of these diagrams will give a fair idea of their completeness. With the Norway iron the curve is fairly well defined up to the breaking point. With the Tennessee charcoal iron the elongation was measured up to the maximum load, at which it was about half the total elongation. No points were obtained on the latter half of the curve. With the Tennessee common iron the curve is well defined up to the maximum load at which the elongation was over four-fifths of the total elongation. No points were obtained on the remainder of the curve. With the plow steel the curve is fairly well defined to the breaking point. With the Bessemer tire steel the curve is well defined to the maximum load at which the elongation was about two-fifths of the total elongation. The breaking load was observed and found to be about three-fourths of the maximum load. Table IV. gives the more important values obtained in these tests.

It will be seen from Table III. that excepting in the case of the plow steel the values of resilience by impact are greater than those obtained under gradual load. The results obtained with the plow steel were irregular and cannot be taken as conclusive. If we leave out the plow steel the gain in resilience by impact ranges from 20.9 to 39.4%, and has an average value of 28.4%. Part of this gain is due no doubt to losses of energy in the impact testing machine. A discussion of these losses appears in the paper already referred to. The error would not exceed 3.5%, it is thought.*

Our experiments, then, would indicate that with wrought iron and soft steel the resilience by impact is decidedly greater than by gradual load. It would appear, also, that this gain by impact is not always of the same value. More data must be obtained upon this point, however, before any safe conclusions may be drawn. This question is a most significant one, as upon it depends very largely the utility of impact tests of steel.

The impact tests above given were made at the office of the St. Louis Water-Works Extension by

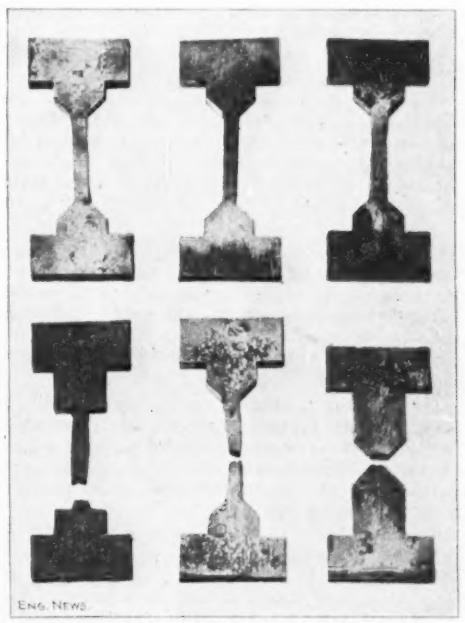

Fig. 3.—View of Impact Tension Test Specimens After Testing.

the writer, assisted by Mr. William F. Schaefer. The gradual load tests were made at Washington University by courtesy of Professor J. L. Van Ornum. The form of test specimen used was first designed in July, 1899. The first test with specimens of this form was not made until December, 1899.

An editorial in the Engineering News of November 16, 1899, suggested independently the use of test specimens of this form and the encourage-

*The theoretical error due to inertia of the fork-bar is about 0.9%, that due to spring of bar is not over 0.8%.

TABLE III.—Showing Comparative Resilience Obtained by Impact and by Gradual Load Tests.

Material.	Thickness of plate in. ins.	By impact. No. of tests.	By impact. Resilience, in.-lbs. per cu. in.	By impact. Elongation in 1-in.	By impact. Reduction of area.	By gradual load. No. of tests.	By gradual load. Resilience in.-lbs. per cu. in.	By gradual load. Elongation in 1-in.	By gradual load. Reduction of area.	Increase by impact.
Iron, Norway.	0.25	4	22,365	0.43	66.6%	2	18,050	0.49	68.1%	23.9%
" Tennessee charcoal	.25	4	17,900	.30	39.1%	2	14,800	.34	43.2%	20.9%
" Tennessee common	.25	4	10,500	.17	19.9%	2	7,530	.20	23.8%	39.4%
Open-hearth plow steel	.30	2	20,800*	.12	....					
	.30	2	10,250	.05†	....	2	20,100	.17	34.5%	....
Soft Bessemer, tire steel	.30	4	28,000	.41	....	2	21,600	.40	....	29.6%

* About. †One test.

TABLE IV.—Giving Detail Results of Gradual Load Tests.

Material.	Specimen No.	Lbs. per sq. in. Maximum strength	Lbs. per sq. in. Limit of elasticity.	Per cent. of Elongation.	Per cent. of Reduction.	Resilience in lbs. per cu. in.
Iron, Norway...	1	38,600	22,700	49.0	65.5	18,050
" " ...	2	39,300	21,200	50.0	70.8	18,050
" Tenn. char.	1	49,600	31,300	36.0	42.0	15,750
" " "	2	49,200	30,600	33.0	44.5	13,800
" " com.	1	44,400	28,300	21.0	25.4	7,300
" " "	2	45,000	25,400	20.0	22.2	7,770
O.-h. plow steel	1	132,400	72,700	15.0	35.3	18,000
" " "	2	127,500	58,500	19.5	33.7	22,270
Bessmr tire st'l	1	57,000	38,200	40.0	....	21,300
" " "	2	58,700	40,000	41.0	....	22,000

ment thus given was not without weight in causing these experiments to be made.

The editorial mentioned also contained certain criticisms of the method of testing steel. It is, perhaps, in order to discuss here the points thus raised. The first point raised by the editor is that there must be a certain loss of energy which is converted into heat at the point of contact with the hammer and knife edges, of which loss no mention was made in the paper under discussion. It is, of course, true that there must be such a loss, but let us go farther and see if we can form an idea of its probable value.

It would appear that where a rounded knife edge of tempered steel rests on a flat surface of the same character, a blow which leaves no mark on the flat surface cannot have produced much movement compared with the stretch of a test specimen of ductile steel. The volume of metal near the point of contact that is distorted appreciably must be very small, and it follows that the elastic resilience of that volume must be a small quantity. The total energy absorbed at the knife edge bearings is, therefore, small comparatively. Now the energy that is converted into heat must be only a part of the total energy represented by the product of the pressure by the distortion. The energy that is converted into heat at the knife edge bearings must then be a small quantity compared with the energy absorbed by the test specimen. We may say, then, that in the absence of direct evidence obtained by properly conducted experiments, we may fairly assume that the energy lost by heat produced at the knife edge bearings does not make a large error in our resilience test.

Another point made by the editor is a criticism of the use of sharp nick test pieces and the expression of the resilience in inch-pounds per square inch. It would seem to the writer that if the steel in a structure is subject to shock, and is of such form that the cross-section is suddenly reduced as at a row of rivet holes in a plate, it would be better to test the steel with a sharp or rounded nick rather than with a long neck of uniform cross-section. The sharp nick is an easy form to make correctly and probably brings out certain qualities of the metal which a prismatic test piece would not show. The writer feels that while the tests with the long neck are a decided step in advance, the tests with the sharp nick should not yet be discarded as of no value.

To return to our proper subject, the experiments given, we may sum up by saying that they indicate that we have here in these tensile impact tests a method of testing steel that gives promise of being useful in the study of the characteristics of that metal. Considered in detail, the experiments indicate that the tensile resilience of soft steel and wrought iron is greater under impact than under gradual load. From the success of these tests and from other considerations the writer believes that larger testing machines could be made on the same plan, and test pieces of greater dimensions could be broken with equal success. The only limitation to the size of the test piece that could be used would be the cost of preparing the specimens.

DEFECTS IN OLD GIRDERS.*

An engineer had occasion very recently to reuse certain old wrought and cast iron girders originally made perhaps 30 to 50 years ago, and his experiences in connection with these old girders may be of interest to others. Some of the cast-iron girders had been in use and carrying railway trains over roads and canals for, say, 40 years, and in consequence of alterations or widenings, the old cast-iron girders had to be removed from the bridges.

It was thought not economical simply to cast aside the old girders as scrap, and therefore a scheme was made to reuse the girders elsewhere, of course leaving an ample margin of safety. Common-sense appeared to point out that materials that had been so long in use under heavy loads would not require testing again before they were reset in another position, and no such provision for testing was made. It was, however, found necessary to drill a few holes through the webs of the old cast-iron girders, in order that tie bolts could be put through them for the attachment of some other parts of the structure. The position of the bolt holes was carefully chosen on the neutral axis, or center of gravity of the section, and an ordinary ratchet brace was fixed to drill the holes. To the astonishment of the workmen who were drilling the hole, no sooner had the drill fairly commenced to drill into the cast iron, than the whole section of the girder snapped across with a loud report.

A second girder was carefully laid on a level floor to avoid any bending action on the girder while it was being drilled, and now again, before the whole was quite through the metal, the second girder broke right across with another loud report. Of course these particular girders and all those that were taken out of that particular bridge were not reused, but were consigned at once to the scrap heap.

In another part of the country certain cast-iron girders were taken out of a bridge that had carried railway engines and trains for many years, and the same girders were re-erected in an overbridge that was to carry comparatively light loads, such as those traveling over bridges in an agricultural district. At all events, the loads were very considerably lighter than those which the girders had previously carried. It was thought that a very satisfactory overbridge had been constructed, and jack arches were built between the girders, and the whole was covered with a good thickness of concrete and asphalt before the top metalling was laid. Scarcely, however, was the bridge finished satisfactorily, and the workmen gone away, than it was found that one of the face girders was cracked right through its section, and this quite close to the abutment bearing. Of course this bridge was pulled down again, and the old girders consigned to the scrap-iron dealer.

Is it a fact that the cast-iron girders, and wrought iron, too, for that matter, of some of our bridges are kept up almost wholly by the "force of habit?" Each of the before-mentioned girders was carefully examined after it had been found broken, but no flaw could be found in the metal. Of course there must have been some reason for this sudden failure, and the only reason suggested was that of unknown internal stresses—due perhaps to the cooling of the metal in the foundry after casting—which possibly the drilling had suddenly relieved in some unknown manned. In some of these old cast-iron girders taken out of bridges carrying heavy loads, and which had been in use for many years, after being carefully examined, flaws of various descriptions have been found. One such girder manifested a slight surface defect. Scraped with a penknife, the scraping revealed a soft and shining surface under the flaw, and suspicions were at once aroused. The girder in question was placed under a powerful hydraulic ram and broken across at the point where the flaw was discovered. Upon examination of the fracture it was found that a pocket at least 4 ins. deep and ¾-in. wide had been filled up with lead—Fig. 1—leaving a surface defect not more than ¼-in. area. Of course many hundred cast-iron girders and columns have been taken down and subsequently reused, and have even been cut and patched, and apparently the result is most satisfactory, but after all the question may very well be asked, Who knows?

The experience recently had with the reuse of some old wrought-iron girders of an age perhaps something near 25 or 30 years is not satisfactory. Wrought iron is a metal very different in behavior during maintenance from cast iron. In the case of wrought iron corrosion goes on very rapidly, and up to a comparatively recent period bridge and roof girders and roof principles were not maintained as such materials now are. The first thing to do with old wrought-iron girders is to examine them carefully as to their corrosion, and the subsequent remarks refer only to girders that had been so examined and saved from the scrap heap by being reported as in "good condition and fit for reuse."

The first case was that of a series of light lattice girders that had been taken down from a railway station roof, where they had been kept constantly painted without any trace of corrosion. Certainly the members were of very slight section, but for a similar position a where it was decided that they were fit for reuse. was, however, found necessary to fix a few new stiff and gussets, and for this purpose the old girders taken to a girder works yard, and several of the rivets were cut out, and some of the plates covering joints also were removed. And then it was found old girderwork is sometimes not equal to the girderw that is now made in our first-class bridge yards. Wh wrought iron was first used it was not considered nec sary to drill the holes, and these were punched. In girders in question it was found that the rivet holes been punched, and very badly and carelessly punch too. The heads before cutting off appeared of corre size and position, but on cutting it was found that holes had been in some cases punched to within ¼-in. the edge of the bar, and this could not be known until was found that the heads were by no means concentr (Fig. 2) with the holes; in fact, nearly all the rivets out had a head altogether on one side of the shank the rivet (Fig. 3). Of course, rivets could not be aga inserted in these holes, and the parts containing the had to be cut away and scrapped. Further, it was found that the butt joints of the flanges which were originally covered by cover plates, were from ½-in. to 1 in. apart, and it was considered that this could not be sanctioned for use as new work. It would have been less troublesome and almost as economical to have ordered new material throughout.

Another case that we have in mind was one where some old wrought-iron girders were taken down from a railway bridge. As these to all appearance were in good condition, it was decided to reuse them in another bridge with a much shorter span, and the girders were cut square across, about one-fourth of the length being so cut away. The ends cut off were taken to a girder maker's yard, and fortunately the foreman immediately set to work to cut them up for scrap purposes. The following points were thereupon discovered, which, let it be noted, could not be and were not discovered at the time that the girder was cut square across. It was found that the whole of the rivets attaching the web plate to the bottom angle iron were practically worn away—not at their heads or visible parts, but inside the work (Fig. 4). Not only these, but the whole of the holes in the web plates had become elongated in shape, the direction of the elongation being at about 45° to the vertical and horizontal lines. The holes were elongated to about double their original size—the original shape, it is assumed, was circular—and the edges were also worn, as seen in Fig. 6.

Further, on examining the bottom flange plate, it was found that the edge of the web plate had cut into the flange plate to the depth of ¼-in. (see Fig. 7), showing

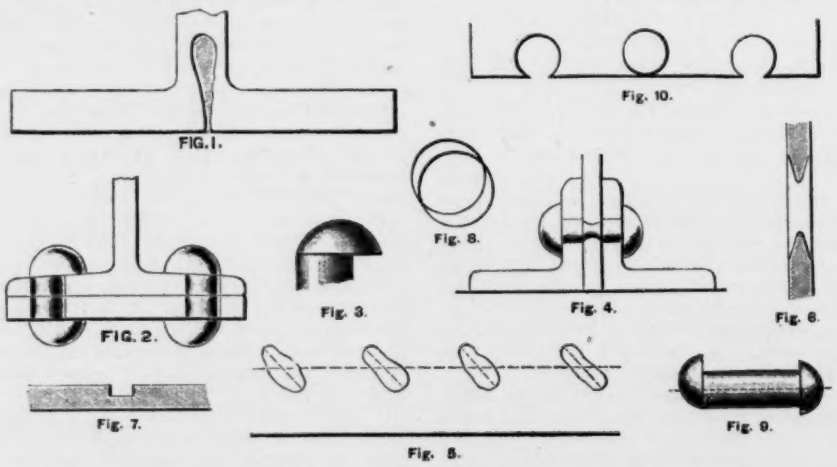

Fig. 5.

*From "The Engineer" (London), Oct. 26.

Hence, after a knowledge of all these points, we venture to suggest that a most careful examination should be made of all old girders that it is proposed to use in new work.